PB82-239054

Handbook for Remedial Action at Waste Disposal Sites

JRB Associates, Inc.
McLean, VA

Prepared for

Municipal Environmental Research Lab.
Cincinnati, OH

Mar 82

U.S. Department of Commerce
National Technical Information Service
NTIS

United States
Environmental Protection
Agency

Office of Emergency and
Remedial Response
Washington DC 20460

Office of Research and
Development
Municipal Environmental Research
Laboratory
Cincinnati OH 45268

Technology Transfer

PB82-239054

EPA

Handbook

Remedial Action at Waste Disposal Sites

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA 22161

TECHNICAL REPORT DATA

(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-625/6-82-006	2.	3. RECIPIENT'S ACCESSION NO. PB82-239054

4. TITLE AND SUBTITLE

Handbook: Remedial Action at Waste Disposal Sites

5. REPORT DATE

March 1982

6. PERFORMING ORGANIZATION CODE

196028

7. AUTHOR(S)

No Credit, See Acknowledgements

8. PERFORMING ORGANIZATION REPORT NO.

9. PERFORMING ORGANIZATION NAME AND ADDRESS

Municipal Environmental Research Laboratory
Office of Environmental Engineering & Technology
Office of Research & Development
U. S. Environmental Protection Agency

10. PROGRAM ELEMENT NO.

TEJY1A

11. CONTRACT/GRANT NO.

68-03-3035

12. SPONSORING AGENCY NAME AND ADDRESS

Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, OH 45268

13. TYPE OF REPORT AND PERIOD COVERED

Final

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

Project Officer - Donald E. Sanning, FTS 684-7871

16. ABSTRACT

This handbook is directed toward technical personnel in federal, state, regional, and municipal agencies involved in the cleanup of hazardous waste disposal sites, industrial surface impoundments, and municipal, industrial, and combined landfills. It contains a summary of the flow of contaminants from a land disposal site, the remedial actions available for site cleanup, and a methodology by which a preliminary remedial action plan can be developed for a specific site. It also includes discussions on remedial actions applicable to surface flow control, groundwater control, leachate control, gas migration control, direct treatment of land disposal wastes, and cleanup of contaminated sewers, water pipes, and sediments. Its objectives are to provide detailed information including a general description, applications, design, construction, and/or operating considerations, advantages and disadvantages, and installation and annual operating costs, with an example where possible. An appendix presents auxiliary information on wastewater treatment unit operations and costs, monitoring methodology, safety and health considerations, and cost update indices.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Attenuation, Barriers, Linings,	Groundwater Pollution	13B
Deactivation, Stabilization, Bentonite	Leachate	68C
Optimization, Waste Disposal Sites	Minimization	89B
Engineering Costs, Design Criteria		

18. DISTRIBUTION STATEMENT Public Distribution	19. SECURITY CLASS *(This Report)* Unclassified	21. NO. OF PAGES 5[illegible]
	20. SECURITY CLASS *(This page)* Unclassified	22. PRICE

EPA Form 2220-1 (Rev. 4-77) PREVIOUS EDITION IS OBSOLETE

EPA-625/6-82-006
June 1982

HANDBOOK FOR REMEDIAL ACTION AT WASTE DISPOSAL SITES

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF ENVIRONMENTAL ENGINEERING AND TECHNOLOGY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

DISCLAIMER

This report has been reviewed by the Office of Environmental Engineering and Technology, U.S. Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

This manual is intended to present information on technologies that may be applicable to specific problems of controlling hazardous wastes at disposal sites. It is not intended to cover any technology exhaustively, nor is the subject of alternative disposal methods addressed except in the context of remedial measures at uncontrolled sites. Neither are the topics of quick- or short-term remedial response actions or management/manifesting procedures considered to be appropriate for inclusion in this manual.

FOREWORD

The Environmental Protection Agency was created because of increasing public and governmental concern about the nation's environment and its effect on the health and welfare of the American people. The complexity of the environment and the interplay among its components require a concentrated and integrated attack upon environmental problems.

The first step in seeking environmental solutions is research and development to define the problem, measure its impact and project possible remedies. Research and development is carried out continually by both industry and governmental agencies concerned with improving the environment. Much key research and development is handled by EPA's Municipal Environmental Research Laboratory. The Laboratory develops new and improved technology and systems, to prevent, treat, and manage wastewater and community sources; to preserve and treat public drinking water supplies; and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research -- a vital communications link between the research and the user community.

The handbook explains the nature of contamination at waste disposal sites and describes some of the remedial actions that can be applied for the clean-up of each contaminated medium. Remedial actions are designed to control, contain, treat or remove contaminants from uncontrolled hazardous waste sites. Remedial actions are divided into surface controls, groundwater controls, leachate controls, direct treatment methods, gas migration controls, techniques for contaminated water and sewer lines, and methods for contaminated sediment removal. The handbook is for industrial and governmental technical personnel involved with the clean-up of uncontrolled hazardous waste sites. In tandem with the proposed National Contingency Plan, it will assist in development of technically sound, environmentally protective, consistent, cost-effective remedies.

Francis T. Mayo, Director
Municipal Environmental Research
Laboratory

William N. Hedeman, Jr., Director
Office of Emergency and Remedial
Response

CONTENTS

Chapter		Page
	FOREWORD	iii
	LIST OF FIGURES	vi
	LIST OF TABLES	xv
	ACKNOWLEDGEMENTS	xxii
1.0	INTRODUCTION	1
	1.1 Background	1
	1.2 Content and Objectives	2
2.0	APPLICATION OF REMEDIAL ACTIONS AT WASTE DISPOSAL SITES	4
	2.1 Introduction	4
	2.2 Environmental Contamination from Waste Disposal Sites	4
	2.3 Selection of Remedial Actions	9
	2.4 Example Site	15
3.0	SURFACE CONTROLS	35
	3.1 Surface Sealing	35
	3.2 Grading	44
	3.3 Revegetation	51
	3.4 Surface Water Diversion and Collection	66
4.0	GROUNDWATER CONTROLS	119
	4.1 Impermeable Barriers	119
	4.2 Permeable Treatment Beds	137
	4.3 Groundwater Pumping	152
	4.4 Interceptor Trenches	182
5.0	LEACHATE COLLECTION AND TREATMENT	192
	5.1 Subsurface Drains	193
	5.2 Drainage Ditches	203
	5.3 Liners	205
	5.4 Leachate Treatment	211
6.0	GAS MIGRATION CONTROL	235
	6.1 Pipe Vents	235
	6.2 Trench Vents	246
	6.3 Gas Barriers	252
	6.4 Gas Collection Systems	258
	6.5 Gas Treatment Systems	269
	6.6 Gas Recovery	277

CONTENTS (Continued)

Chapter — Page

7.0 DIRECT WASTE TREATMENT METHODS 283
  7.1 Excavation 284
  7.2 Hydraulic Dredging 295
  7.3 Land Disposal 310
  7.4 Solidification 313
  7.5 Encapsulation 321
  7.6 In-Situ Treatment 326
  7.7 Other Direct Treatment Techniques 334

8.0 CONTAMINATED WATER AND SEWER LINES 339
  8.1 In-Situ Cleaning 339
  8.2 Leak Detection and Repairs 342
  8.3 Removal and Replacement 345
  8.4 Costs 345
  8.5 Monitoring 347

9.0 CONTAMINATED SEDIMENTS 353
  9.1 Mechanical Dredging 354
  9.2 Low-Turbidity Hydraulic Dredging 367
  9.3 Dredge Spoil Management 371
  9.4 Revegetation 384

APPENDIX A - Monitoring Systems 391
APPENDIX B - Wastewater Treatment Modules 405
APPENDIX C - Cost Indices 494

COPYRIGHT NOTICE 496

LIST OF FIGURES

Figure		Page
2-1	Initial Transport Processes at Waste Disposal Sites	6
2-2	Hydrologic Pathways for Contamination by Waste Disposal Sites	7
2-3	Flow of Land-Disposed Waste Contaminants through the Environment	8
2-4	Flow Chart for Selection of Remedial Actions	11
2-5	Remedial Action Decision Flow for Hypothetical Waste Disposal Site	29
2-6	Characteristics of Site I	31
3-1	Fly Ash Sources East of Rocky Mountains	39
3-2	Two Typical Layered Cover Systems	41
3-3	Grading Vehicles and Accessories	49
3-4A	Cross Section End View of Gas Barrier Trench	62
3-4B	Cross Section End View of Soil Mound	62
3-5	Temporary Interceptor Dike	68
3-6	Temporary Diversion Dike	69
3-7	Typical Drainage Ditch at Base of Disposal Site	72
3-8	Effect of Drainage Ditch Shape on Velocity and Erosivity	74
3-9	Standard Design for Drainage Ditches	77
3-10	General Design Features of Diversions	78
3-11	Grassed Waterways with Stone Centers	79

LIST OF FIGURES (Continued)

Figure		Page
3-12	Grassed Waterways	80
3-13A	Bench Terraces with Reserve Fall	82
3-13B	Bench Terraces with Natural Fall	82
3-14	Typical Drainage Bench	83
3-15	Slope Reduction Measures	84
3-16	Half-Round Bituminous Fiber Pipe used for Temporary Handling of Concentrated Flow	87
3-17	Paved Chute (or Flume)	89
3-18	Rigid Downpipe	91
3-19	Flexible Downdrain	92
3-20	Typical Levee at Base of Disposal Site	95
3-21	Perimeter Flood Protection Structure	95
3-22A	Levee with Impervious Core	96
3-22B	Levee with Cutoff and Drain	96
3-23	Some Typical Floodwall Sections	98
3-24	Seepage Basin; Large Volume, Deep Depth to Groundwater	100
3-25	Seepage Basin; Shallow Depth to Groundwater	100
3-26	Seepage Ditch	101
3-27	Seepage Ditch with Increased Seepage Efficiency	102
3-28	Particle Size Distribution Graph	105
3-29	Typical Design of a Sediment Basin Embankment	106
3-30	Modification of Conventional Sedimentation Pond to Reduce Suspended Solids	109
3-31	Integrated System of Interception and Diversion Techniques	111
3-32	Level Spreader	112

LIST OF FIGURES (Continued)

Figure		Page
4-1	Typical Two-row Grid Pattern for Grout Curtain	126
4-2	Semicircular Grout Curtain Around Upgradient End of Landfill	132
4-3	Some Steel Piling Shapes and Interlocks	134
4-4	Installation of a Permeable Treatment Bed	140
4-5	pH Changes with Contact Time	143
4-6	Lowering a Water Table to Prevent Stream Dicharge of Contaminated Water	153
4-7	Lowering the Water Table to Eliminate Contact with a Disposal Site	154
4-8	Lower Water Table to Prevent Contamination of an Underlying Aquifer	155
4-9	Schematic of a Well Point Dewatering System	156
4-10	Hydraulic Conductivities of Soil and Rock	158
4-11	Drawdown in Potentiometric Surface of a Confined Aquifer Being Pumped by Two Wells	161
4-12	Use of Extraction/Injection Wells for Plume Containment	169
4-13	Groundwater Pumping to Contain a Plume (No Recharge)	170
4-14	Use of Extraction Wells for Plume Containment Followed by Subsequent Recharge Through Seepage Basins	171
4-15	Effect of an Injection Well on the Cone of Depression	174
4-16	Flow Scheme of TACION Apparatus for Industrial Applications	180
4-17	Treatment of the Contaminated Groundwater with the Bioreclamation Technique	186
5-1	Subsurface Leachate Collection Drain	195
5-2	Diagram for Hooghoudt's Drain-Spacing Formula	196
5-3	Design Plan for Leachate Collection System at Love Canal	200
5-4	Leachate Collection System for Love Canal - Traverse View	201

LIST OF FIGURES (Continued)

Figure		Page
5-5	Leachate Collection Costing Example	210
5-6	Changes in Ratios of Various Parameters with the Age of the Landfill	213
5-7	Schematic of Leachate Treatment System at GROWS Landfill	226
5-8	Biochemical Oxygen Demand of Leachate	230
5-9	Total Organic Carbon Concentration of Leachate	231
5-10	Chemical Oxygen Demand of Leachate	232
6-1	Design Configuration of Pipe Vents	237
6-2	Uniform Distribution of Methane Gas in Landfill	240
6-3	Non-Uniform Distribution of Methane Gas in Landfill	241
6-4	Variation of Effectiveness of Vents	241
6-5	Discharge Rating Curve	243
6-6	Head Loss Curve	244
6-7	Design Configurations of Trench Vents	248
6-8	Design of Anchor Trench	256
6-9	Fan Design Chart	261
6-10	Nomograph for Solution of Friction Losses in Pipe	262
6-11	Elbow and Branch Take-off Losses	265
6-12	Example of Collection System	266
6-13	Capital and Operating Costs for Non-regenerative Carbon Adsorption Systems Treating Vent Gas Containing 50 ppm Trichloroethylene	276
6-14	Total Installed Cost for Small Ground Flares	278
7-1	A Dragline	286
7-2	Typical Working Ranges for Cranes and Draglines	288

LIST OF FIGURES (Continued)

Figure		Page
7-3	Typical Backhoe	290
7-4	Hoe Digging Ranges	291
7-5	The Clamshell Bucket	293
7-6	Portable Centrifugal Pump System for Lagoon Dredging	297
7-7A	Standard Cutter Assembly: Spiral Basket Cutter	299
7-7B	Standard Cutter Assembly: Spiral Basket Cutter	299
7-8	The Standard Cutterhead Dredge Vessel	300
7-9	Standard Cutterhead Dredge Operation	300
7-10	Views of the Mud Cat MC-915 Dredge	301
7-11	Important Distances in the Selection of Dredging Systems Equipment	304
7-12	Typical Capacity Chart for Portable Cutterhead Dredge	305
7-13	Encapsulation Process Concept	323
7-14	Schematic of Cementation/Encapsulation Process for Containment of Hazardous Wastes	324
7-15	In-Situ Detoxification Process for Cyanide	330
8-1	Power Rodding Machine	341
8-2	Schematic of Bucket Machine Cleaning	342
8-3	Interior Relining with PE	344
9-1	Streamflow Diversion for Sediment Excavation Using Two Cofferdams and Diversion Channel	356
9-2	Streamflow Diversion for Sediment Excavation Using Single Cofferdam	357
9-3	Construction of a Typical Center Tension Silt Curtain Section	358
9-4	Typical Silt Curtain Deployment Configurations	359

LIST OF FIGURES (Continued)

Figure		Page
9-5	The Japanese Suction Dredge "Clean Up"	369
9-6	Dredging Rates to Transport Various Materials	375
9-7	Typical Grain-Size Distributions for Dredged Material	376
9-8	Portable Contaminated Spoil Processing System	378
9-9	Construction Cost of Dredge Spoil Containment Basin	385
A-1	Typical Monitor Well Network, Areal View	393
A-2	Typical Monitoring Well Screened Over a Single Vertical Interval	395
A-3	Typical Well Cluster Configuration	396
A-4	Surface Water Sampling Equipment	400
A-5	Typical Gas Probe Placement	402
A-6	Typical Multi-Level Gas Sampling Probe Installation	403
B-1	1976 Construction Costs for Concrete Equalization	407
B-2	1976 O&M Costs for Concrete Equalization Basins	407
B-3	Representative Configuration Employing Precipitation, Flocculation, and Sedimentation	409
B-4	Typical Solids Contact Chemical Treatment	409
B-5	1976 Construction and O&M Costs for a Primary Clarifier	415
B-6	Costs for Lime Storage, Handling, and Feed (1976 Costs)	416
B-7	Costs for Alum Storage, Handling, and Feed (1976 Costs)	417

LIST OF FIGURES (Continued)

Figure		Page
B-8	Construction Cost for Package Treatment Plants at Filtration Rates of 2 and 5 gpm/ft (1977 Costs)	419
B-9	Operation and Maintenance Requirements for Package Complete Treatment Plants - Labor and Total Costs at Filtration Rates of 2 and 5 gpm/ft (1977 Costs)	420
B-10	Typical Package Water Treatment Plant for Precipitation, Flocculation, and Sedimentation	421
B-11	Construction Costs for Conventional Activated Sludge Treatment (1976 Costs)	432
B-12	O&M Costs for Conventional Activated Sludge Treatment (1976 Costs)	432
B-13	Construction Costs for An Extended Aeration Package Plant (1976 Costs)	433
B-14	O&M Costs for an Extended Aeration Package Plant	433
B-15	Construction Costs for Pure Oxygen-Activated Sludge	434
B-16	O&M Costs for Pure Oxygen-Activated Sludge	434
B-17	Construction Costs for Rectangular Clarifiers Secondary Clarifiers (1976 Costs)	435
B-18	O&M Costs for Rectangular Secondary Clarifiers	435
B-19	Trickling Filter Recirculation	438
B-20	Construction Costs for Trickling Filter Package Plant (1976 Costs)	440
B-21	O&M Costs for Trickling Filter Package Plant (1976 Costs)	440
B-22	Bio-Surf Process Schematic	440

LIST OF FIGURES (Continued)

Figure		Page
B-23	Construction Costs for Rotating Biological Discs	443
B-24	O&M Costs for Rotating Biological Discs	443
B-25	1976 Construction Costs for Anaerobic Lagoons	449
B-26	O&M Costs for Anaerobic Lagoons	449
B-27	Construction Costs for Facultative Lagoons (1976 Costs)	451
B-28	O&M Costs for Facultative Lagoons (1976 Costs)	451
B-29	Construction Costs for Aerated Lagoons	452
B-30	O&M Costs for Aerated Lagoons	452
B-31	Adsorber Configurations	457
B-32	Construction Costs for Tertiary Activated Carbon Treatment (1976 Costs)	461
B-33	O&M Costs for Tertiary Activated Carbon Treatment (1976 Costs)	461
B-34	Mixed Bed Operating Cycle Showing (A) Service Period, (B) Backwash with Resin Segregation, (C) Regeneration, and (D) Resin Re-Mixing	466
B-35	Cyclic Multibed System, Each Step Having Two Beds in Process Use (1 Ahead of 2) and One Bed Offstream for Regeneration	466
B-36	Liquid Ion Exchange of Metal Finishing Wastes	474
B-37	Ammonia Stripping Lagoon	478
B-38	Ammonia Stripping Tower	478
B-39	Schematic of Ammonia Removal and Recovery System	479

LIST OF FIGURES (Continued)

Figure		Page
B-40	Capital Costs of Ammonia Stripping System, Including Related Yardwork Engineering, Legal, Fiscal, and Financing Costs During Construction and Excluding Cost of pH Adjustment. Based on 1 gpm/sf of Tower Packing 24 ft Packing Depth. Includes Influent Pumping (TDH = 50 ft).	480
B-41	Operation and Maintenance Costs of Ammonia Stripping System, Excluding Cost of pH Adjustment; Labor Fixed @ 59/Hour, Power @ 50.02/kwh.	480
B-42	Wet Air Oxidation Flow Schematic	481
B-43	Extent of Hypochlorous Acid (HOCl) Ionization into OCL⁻ as a Function of pH and Temperature	488
B-44	Construction Costs for Chlorination	490
B-45	O&M Costs for Chlorination	490

LIST OF TABLES

Table		Page
2-1	Important Site Characteristics and Considerations Affecting Selection of Remedial Measures	13
2-2	Summary of Available Remedial Action Techniques for Contaminated Surface Flows	16
2-3	Summary of Available Remedial Action Techniques for Contaminated Groundwater	19
2-4	Summary of Available Remedial Action Techniques for Contaminated Air/Soil Pore Space	22
2-5	Summary of Available Remedial Action Techniques for Contaminated Soil and Sediments	23
2-6	Summary of Available Remedial Action Techniques for Hazardous Wastes	25
2-7	Summary of Available Remedial Action Techniques for Contaminated Water and Sewer Lines	28
3-1	Ranking of Soil Types Based on Percolation Control and Resistance to Wind Erosion	36
3-2	Estimated Unit Costs for Surface Sealing Methods and Materials	45
3-3	Cover Material Handling Characteristics of Landfill Equipment	48
3-4	Unit Costs Associated with Grading Covered Disposal Sites	51
3-5	Characteristics of Grasses and Legumes Commonly Used for Revegetation	53
3-6	Commonly Used Trees and Shrubs	55
3-7	Summary of Chemical Stabilizers for Cover Soil	59
3-8	Unit Costs Associated with Revegetation of Covered Disposal Sites	64

LIST OF TABLES (Continued)

Table		Page
3-9	Permissible Design Velocities for Stabilized Diversions and Waterways	75
3-10	Bottom Widths and Maximum Drainage Areas for Temporary Chutes	88
3-11	Summary of Advantages and Disadvantages of Chutes and Downpipes	93
3-12	Summary Evaluation of Levees	99
3-13	Advantages and Disadvantages of Gravity Drainage Systems	102
3-14	Design Storage Capacity Requirements for Sedimentation Basins	108
3-15	Unit Costs Associated with Surface Water Diversion and Collection Structures	114
4-1	Permeability Increase due to Leaching with Various Pollutants	122
4-2	Costs Associated with Slurry Wall Construction	124
4-3	Approximate Cost of Grout	130
4-4	Costs Associated with Silicate Grout Curtain Placement	131
4-5	Advantages and Disadvantages of Sheet Piling Wall	135
4-6	Unit Cost for Installation of a Sheet Piling Wall	136
4-7	Results of Chemical Analyses of Greensand Filtration of Pigeon Point Landfill Leachate	145
4-8	Advantages and Disadvantages of Crushed Limestone Treatment Bed	146
4-9	Advantages and Disadvantages of Activated Carbon Treatment Bed	146
4-10	Advantages and Disadvantages of Glauconitic Treatment Bed	148
4-11	Unit Costs for Installation of a Permeable Treatment Bed	148
4-12	Values of $W(\mu)$ for Various Values of μ	160

LIST OF TABLES (Continued)

Table Page

4-13 Advantages and Disadvantages of Well Point Systems for Water Table Adjustments 166

4-14 Unit Costs for Equipment Used in Well Point Dewatering 168

4-15 Advantages and Disadvantages of Groundwater Pumping to Contain a Plume 177

4-16 Costs for Well Construction and Installation 178

4-17 Advantages and Disadvantages of the Groundwater Treatment System 183

4-18 Costs for Physical-Chemical Treatment Systems 183

4-19 Advantages and Disadvantages of the Bioreclamation Technique 187

5-1 Advantages and Disadvantages of Subsurface Drains 202

5-2 Advantages and Disadvantages of Drainage Ditches 204

5-3 Advantages and Disadvantages of Liners at Existing Sites 207

5-4 Unit Costs for a Leachate Collection System 208

5-5 Composition of Leachate from Different Sources as Measured at the University of Illinois (UI) 214

5-6 Summary of Leachate Treatment Processing Units 216

5-7 Cost Comparison for Treatment Modules (0.1 and 0.3 mgd) 224

5-8 Treatment Performance After Acclimation of Activated Sludge (August 1977) 227

5-9 Costs for Municipal Leachate Treatment System 229

6-1 Unit Costs for Installation of Trench Units 250

6-2 Chemical Resistance Chart (DuPont) 255

6-3 Costs for Gas Barriers 257

6-4 K Values for Fittings and Valves 263

6-5 Unit Costs of Components for Gas Collection Systems 268

LIST OF TABLES (Continued)

Table		Page
6-6	Retentivity by Activated Carbon	273
6-7	Advantages and Disadvantages of Carbon Adsorption Gas Treatment Systems	274
6-8	Advantages and Disadvantages of Flare Systems	274
6-9	Advantages and Disadvantages of Afterburners	275
6-10	Costs for Landfill Gas Recovery at Mountain View, California	279
7-1	Typical Dragline Excavator Dimensions	287
7-2	Theoretical Hourly Production Rate of a Dragline Excavator	289
7-3	Maximum Reach and Depth for Various Sized Hoes	292
7-4	Theoretical Hourly Production of a Hydraulic Backhoe	292
7-5	Advantages and Disadvantages of Different Types of Excavators	294
7-6	Unit Cost for Excavation	296
7-7	Summary of Evaluation of Hydraulic Dredging of Surface Impoundments	307
7-8	Unit Costs for Hydraulic Dredging of Surface Impoundments	308
7-9	Costs of Potential Solution Mining Chemicals	328
7-10	Costs for In-Situ Detoxification of Cyanide	331
7-11	Costs of Potential In-Situ Neutralization/Detoxification Chemicals	331
7-12	Unit Costs for In-Situ Microbial Degradation	333
8-1	Costs for Cleaning, Repair, and Replacement of Water and Sewer Lines	346
9-1	Summary of Evaluation of Instream Mechanical Dredging of Contaminated Sediments	363
9-2	Summary Evaluation of Cofferdam Method for Sediment Removal	363

LIST OF TABLES (Continued)

Table		Page
9-3	Costs for Mechanical Dredging/Excavation of Contaminated Sediments	363
9-4	Unit Costs Associated with Dredge Spoil Management	381
9-5	Unit Costs for Dredged Marshland Restoration Activities	387
A-1	Cost Estimates for Various Monitoring Techniques and Construction Methods in the Zone of Saturation	397
B-1	Approximate Solubilities of Metals (ppm in Pure Water)	411
B-2	Chemical Treatment of Industrial Wastewater by Coagulation	412
B-3	Advantages and Disadvantages of Precipitation, Coagulation, and Flocculation	414
B-4	Construction Cost for Package Complete Treatment Plants	422
B-5	Operation and Maintenance Summary for Package Complete Treatment Plants	423
B-6	Threshold Concentration for Various Metals in the Air-Activated Sludge Process	425
B-7	Summary of Operating Parameter for Air-Activated Sludge and Pure Oxygen-Activated Sludge	427
B-8	Advantages and Disadvantages of Activated Sludge and Pure Oxygen Activated Sludge	431
B-9	Design Criteria for Trickling Filters	437
B-10	Advantages and Disadvantages of Trickling Filters	438
B-11	Advantages and Disadvantages of Rotating Biological Discs	442
B-12	Advantages and Disadvantages of Treatment with Acclimated Bacteria	445

LIST OF TABLES (Continued)

Table		Page
B-13	Major Design Criteria for Lagoons	447
B-14	Advantages and Disadvantages of Stabilization Ponds and Aerated Lagoons	448
B-15	Effects of Molecular Structure and Other Factors on Adsorption by Activated Carbon	454
B-16	Potential for Removal of Inorganic Material by Activated Carbon	455
B-17	Summary of Activated Carbon Contacting Methods	458
B-18	Operating Parameters for Carbon Adsorption	459
B-19	Advantages and Disadvantages of Activated Carbon	460
B-20	Common Reactive Groups for Ion Exchange Resin	464
B-21	Comparison of Ion Exchange Operating Modes	465
B-22	Examples of Regenerants and Dosage Ranges	467
B-23	Advantages and Disadvantages of Ion Exchange	468
B-24	Conceptual Design for Pressure Ion Exchange Nitrate Removal	469
B-25	1978 Construction Cost for Pressure Ion Exchange Nitrate Removal	470
B-26	Operation and Maintenance Summary for Pressure Ion Exchange Nitrate Removal	471
B-27	Ion Exchange Cost Estimate (1976 Dollars)	472
B-28	Advantages and Disadvantages of Liquid Ion Exchange	475
B-29	Liquid Ion Exchange Cost Example Operating Cost Estimates for Liquid Ion Exchange (1976 Costs)	476

LIST OF TABLES (Continued)

Table		Page
B-30	Advantages and Disadvantages of Ammonia Stripping	479
B-31	Wet Air Oxidation of Toxic Organics	483
B-32	Advantages and Disadvantages of Wet Air Oxidation	484
B-33	Feed Waste Characteristics Assumed for Cost of Wet Air Oxidation	485
B-34	Performance/Operating Criteria for Wet Air Oxidation	485
B-35	Cost of WAO (1979 Dollars)	486
B-36	Advantages and Disadvantages of Chlorination	489

ACKNOWLEDGEMENTS

JRB Associates, Inc. (JRB) prepared this document for EPA's Office of Research and Development (ORD) in partial fulfillment of Contract No. 68-01-4839, Directive of Work No. 15. Mr. Paul Des Rosiers of ORD was Project Officer and Mr. Michael Kilpatrick of the Hazardous Waste Enforcement Task Force and Mr. John Frisco of EPA Region II were co-Project Monitors. Mr. Robert Colonna was JRB's Project Manager. Principal contributors include Mr. Paul Rogeshewski, Mr. Hal Bryson, and Ms. Kathleen Wagner. Other contributors were Mr. Phouc Le, Mr. Phillip Spooner, Dr. Sidney Paige, and Mr. David Twedell. Mr. Donald Sanning, Municipal Environmental Research Laboratory, Cincinnati, Ohio and Mr. William Librizzi, Director of the Survey and Analysis Division, EPA Region VI, are acknowledged for their valuable contributions and as technical reviewers. The efforts and encouragement of Richard Stanford in coordinating activities between the Office of Emergency and Remedial Response (OERR) and the Office of Research and Development (OR&D) are acknowledged.

A peer review of the final draft was carried out by a team under the organizational management of WAPORA, Inc., in partial fulfillment of Contract No. 68-01-5913, Work Assignment No. WA-7. Members of the team were associated with: 1) WAPORA, Inc.; 2) Best Environmental Service and Technology, Ltd.; 3) Environmental Resources Management, Inc.; 4) Midland-Ross Corporation; and 5) Rollins Environmental Services. Comments and suggestions relative to the final draft were submitted as a separate report to ORD/OEET of the U.S. EPA. This report was then reviewed by the U.S. EPA and the final draft was modified to incorporate the recommendations that were approved by these reviewers.

1.0 INTRODUCTION

1.1 BACKGROUND

Improper disposal of our nation's municipal, commercial, and industrial solid and hazardous wastes is one of the most pressing environmental problems. The quantity of the nation's waste generated and disposed of annually is tremendous and growing. EPA estimates the amount of these wastes at:

- 59 million metric tons of hazardous waste generated annually.
- 33 million metric tons of municipal wastes disposed of annually in 18,500 sites covering a total of 500,000 acres.
- 5 million metric tons of municipal wastewater treatment sludge generated annually; the amount will double within this decade because of higher treatment levels.
- Over 120 million metric tons of flue gas cleaning sludges will be generated annually by 1985, enough to cover an area 10 square miles with a 9-foot sludge layer.
- Billions of tons of agricultural and mining wastes generated yearly.

The environmental consequences of improper disposal of solid and hazardous wastes have already been demonstrated by such episodes as New York's Love Canal and Kentucky's Valley of the Drums. Similar cases of improper waste management have resulted in contamination of local groundwater, surface water, land and air, food and forage crops.

In response to the negative impacts of improper waste disposal, Congress passed the Resource Conservation and Recovery Act (RCRA) in 1976. RCRA addresses all aspects of waste management, but most significantly, managing hazardous wastes. A "cradle to grave" management concept has been adopted to regulate generation, transportation and disposal of hazardous waste. In 1980, legislation was passed to provide a $1.6 billion "Superfund" for the clean-up of inactive and abandoned hazardous waste disposal facilities. Sections of other acts, such as the Clean Water Act, the Safe Drinking Water Fund, and the Rivers and Harbors Act also provide vehicles for cleaning up waste disposal sites.

As a result of clean-up operations that have already been conducted at such sites, and in anticipation of site clean-up activities that will result from RCRA and Superfund activities, many technologies have and are being developed. Technologies designed specifically for clean-up of waste disposal sites have been termed "remedial actions." Such actions at a waste disposal site may take a variety of forms. They may consist of surface flow controls that divert and channel rainfall, thus preventing infiltration of water into the waste site. They may control the spread of contaminated groundwater, either by containment or pumping. Other types of remedial actions involve controlling the migration of dangerous gases and vapors from the site, removing the waste material from the site for treatment or disposal, and cleaning up water mains, sewers, wetlands, and water bodies that have been contaminated by wastes at that site.

This handbook describes the available remedial action technologies and how they may be selected and applied for the clean-up of waste disposal sites. Particular emphasis is placed on clean-up at hazardous waste sites. Since hazardous wastes are often disposed of near or in combination with municipal refuse and other solid wastes, any remedial actions specificially applicable to problems imposed by these other wastes have also been included in the handbook discussions.

1.2 CONTENT OF OBJECTIVES

This is a technical handbook to assist in the development of remedial action plans for the clean-up of waste disposal sites. The handbook is geared primarily for technical personnel in federal, state, regional, and municipal agencies involved in the clean-up of hazardous waste disposal sites, industrial surface impoundments, and municipal, industrial, and combined landfills. The manual is divided into three major parts:

(1) Chapter 2 presents a summary of the flow of contaminants from a land disposal site, the remedial actions available for site clean-up, and a methodology by which a preliminary remedial action plan can be developed for a specific site. The chapter's objectives are to:

- o Provide a basic understanding of pollutant pathways involved in a waste disposal site

- Provide a basic understanding of the remedial actions and how they apply to site characteristics such as climate, hydrology, type of waste, etc.
- Provide a generalized approach to assist the process of selecting remedial actions for a polluting site
- Provide an example of the selection of remedial actions for a specific site.

(2) Chapters 3-9 provide the reader with detailed information on remedial actions applicable to surface flow control, ground water control, leachate control, gas migration control, direct treatment of land disposal wastes, clean-up of contaminated sewers and water pipes, and clean-up of contaminated sediments. The objectives of each section are to provide detailed information on each remedial option, including:

- A general description
- Applications
- Design, construction, and/or operating considerations
- Advantages
- Disadvantages
- Installation and annual operating costs, with an example where possible.

(3) An appendix presents auxiliary information on wastewater treatment unit operations and costs, monitoring methodology, safety and health considerations, and cost update indices. This information is considered essential to the development of a remedial action plan; it was put in appendix form because it is largely tabular and because it applies to all of the options in Chapters 3-9.

2.0 APPLICATION OF REMEDIAL ACTIONS AT WASTE DISPOSAL SITES

2.1 INTRODUCTION

This chapter begins with a discussion of environmental contamination resulting from polluting waste disposal sites. It then provides a methodology for screening remedial actions on the basis of specific site characteristics and for developing a preliminary remedial action plan. Chapter 2 is for use prior to and in conjunction with the detailed technical sections that follow (Chapters 3-9) so that the best remedial approaches can be evaluated on the basis of technical feasibility, performance, maintenance, service life, and costs.

Section 2.2 discusses the overall transport of land-disposed waste contaminants through the various environmental media to potential receptors. It is intended to provide the reader with an understanding of the many different pathways a pollutant may follow and why remedial actions at a waste disposal site must offer complete control.

Section 2.3 provides a methodology for evaluation of remedial actions on the basis of specific site characteristics. It contains a number of tables in which remedial actions are classified with respect to the media they are intended to control as well as their applicability to specific site characteristics.

The remedial action evaluation process can often be arduous, and Section 2.4 is designed to give the reader a better feel for the complications that can arise in selecting remedial actions. The section presents an example site, for which the information provided in this chapter is used to develop preliminary remedial action recommendations. The example illustrates that, although the methodology is very useful, it cannot always be regarded as a "cookbook" for remedial action selection.

2.2 ENVIRONMENTAL CONTAMINATION FROM WASTE DISPOSAL SITES

Land-disposed waste materials, whether disposed in landfills, surface impoundments, or other types of land disposal facilities, are subject to various transport processes that may lead to environmental contamination. These transport processes involve an initial transformation to a more mobile phase, usually by solubilization, volatilization, or a chemical or biochemical reaction to form soluble or gaseous reaction products. Figure 2-1 presents an overall view of the initial mobilization processes involved in pollutant transport. It can be seen from the figure that volatile and water-soluble components are formed from microbial degradation and chemical reactions with other wastes. Often, the chemical and biochemical reactions can end in explosions and fires that emit particulates, as well as combustion products, to the atmosphere. Particulates can also be entrained by surface runoff coming into contact with the waste material. The figure illustrates that wastes have the potential to be mobilized in any phase, given the "right" conditions.

When transport mechanisms are available, the waste materials may migrate outside a disposal site and pollute the groundwater, surface water, air, and terrestrial and benthic environments. Water plays an especially important role in the mobilization and transport of waste materials from the disposal site to the environment; therefore, the hydrologic cycle should be well understood as it relates to the geology and topography of a disposal site. Figure 2-2 illustrates the flow of water through and around a waste disposal site and the transport of waste constituents to various environmental receptors. It is evident that there are many potential hydraulic pathways a contaminant may follow, depending on site characteristics. For example, leachate may travel downward vertically to contaminate groundwater, or it may travel laterally and emerge as surface seepage, depending on local soil characteristics. It is important to recognize the hydraulic relationship between groundwater and surface water, and to realize that either can contaminate the other, given the right conditions.

The environmental effects resulting from polluting land disposal sites can be localized or widespread, direct or indirect, apparent or obscure, short-term or long-term. Figure 2-3 illustrates the flow of land-disposed waste contaminants through the environment and to various receptors. Obvious effects of a polluting disposal site may include contamination of a local drinking water well by a contaminated water table, direct inhalation of waste fumes by nearby workers and residents, or contamination of surface water by a leachate surface seep. On the more obscure side, contamination of an aquatic food chain may occur via biological uptake of settled wastes by benthic (sediment) organisms. Also, a major municipal water supply aquifer may experience a slow long-term degradation in water quality, or an explosion in a basement may occur as a result of lateral methane migration from a landfill closed years ago.

The establishment of an effective remedial action plan for a polluting waste disposal site must take into account all of the pathways involved in the transport of contaminants through the environment and to receptors. Remedial actions may be taken on several levels: (1) direct treatment or removal of the land-disposed wastes, (2) prevention of waste migration from the site, (3) clean-up of affected media (contaminated sediments or sewers, (4) relocation and/or upgrading of water supply systems, and (5) litigation and mitigation such as a fishing ban, closing of a school or the payment of relocation costs to protect and/or compensate affected populations.

FIGURE 2-1

INITIAL TRANSPORT PROCESSES AT WASTE DISPOSAL SITES

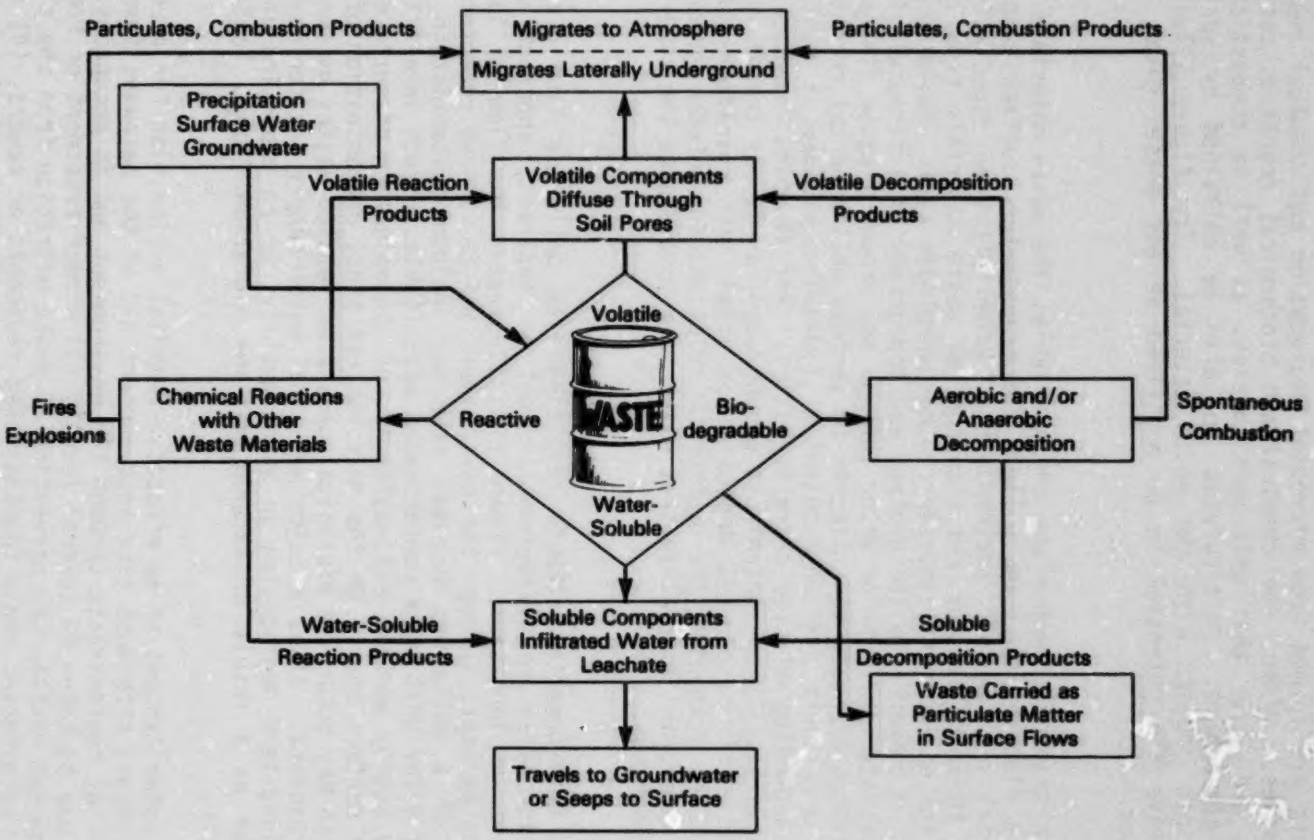

Particulates, Combustion Products
Migrates to Atmosphere
Migrates Laterally Underground
Particulates, Combustion Products
Precipitation
Surface Water
Groundwater
Volatile Reaction
Products
Volatile Components
Diffuse Through
Soil Pores
Volatile Decomposition
Products
Volatile
Fires
Explosions
Chemical Reactions
with Other
Waste Materials
Reactive
WASTE
Bio-
degradable
Aerobic and/or
Anaerobic
Decomposition
Spontaneous
Combustion
Water-
Soluble
Water-Soluble
Reaction Products
Soluble Components
Infiltrated Water from
Leachate
Soluble
Decomposition Products
Waste Carried as
Particulate Matter
in Surface Flows
Travels to Groundwater
or Seeps to Surface

FIGURE 2-2

HYDROLOGIC PATHWAYS FOR CONTAMINATION BY WASTE DISPOSAL SITES

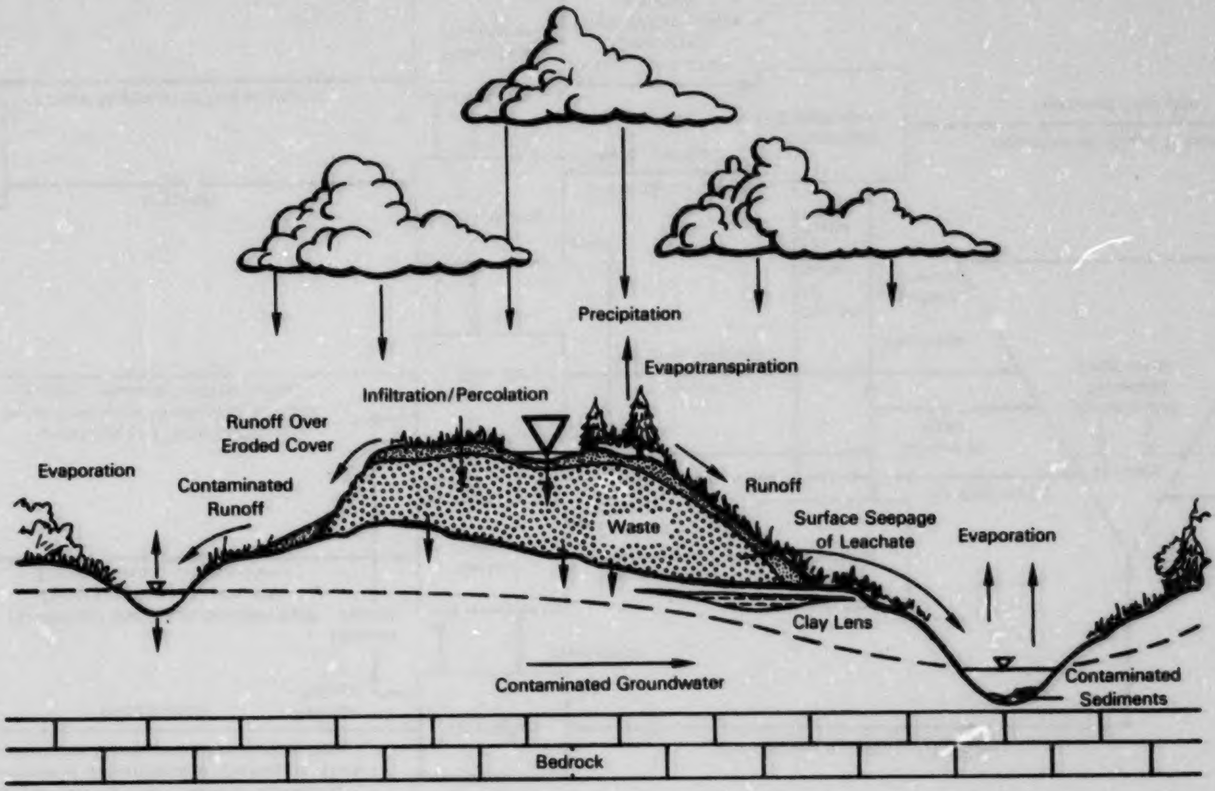
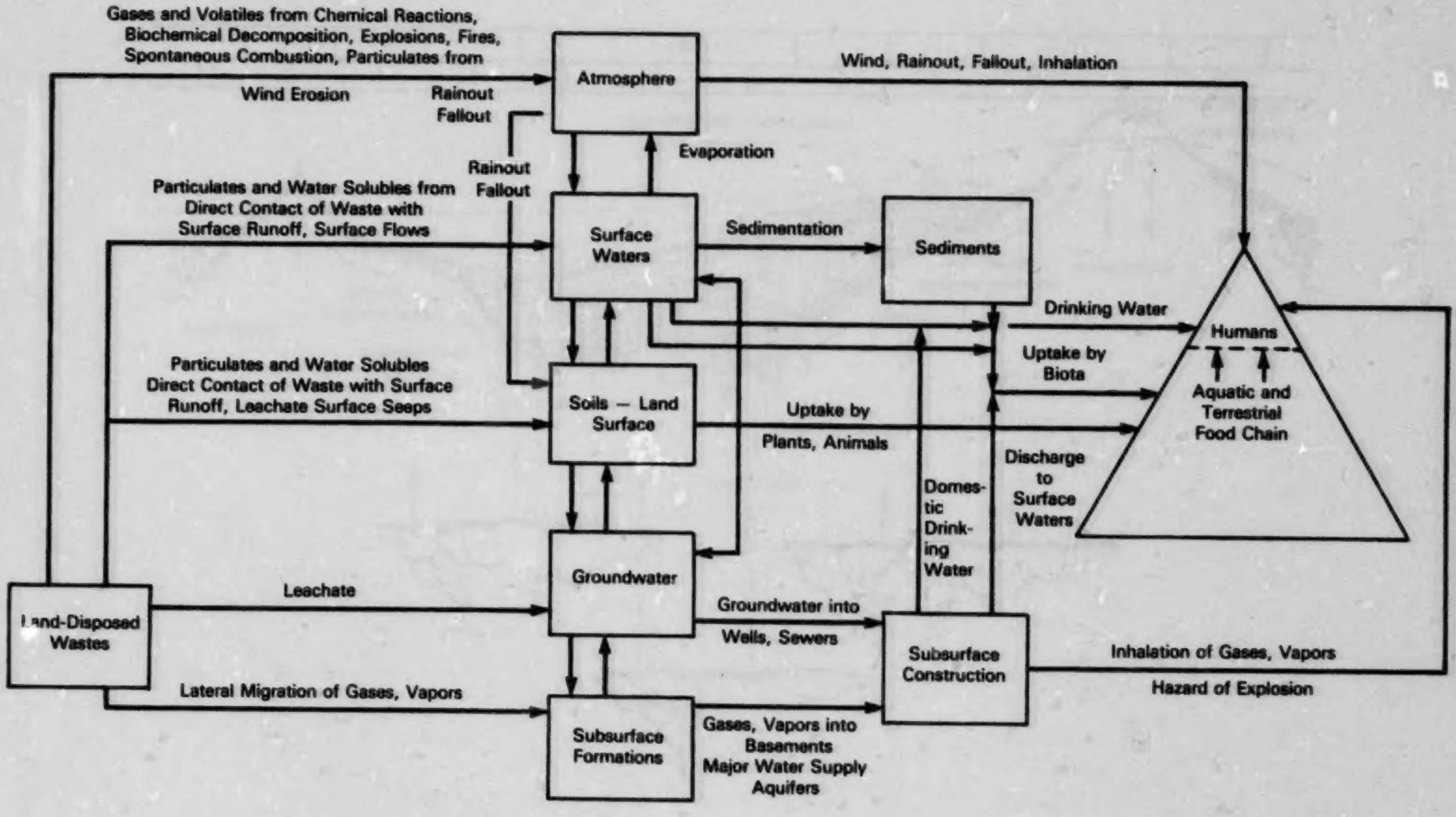

Precipitation
Evapotranspiration
Infiltration/Percolation
Runoff Over Eroded Cover
Evaporation
Contaminated Runoff
Runoff
Waste
Surface Seepage of Leachate
Evaporation
Clay Lens
Contaminated Groundwater
Contaminated Sediments
Bedrock

FIGURE 2-3

FLOW OF LAND-DISPOSED WASTE CONTAMINANTS THROUGH THE ENVIRONMENT


Gases and Volatiles from Chemical Reactions, Biochemical Decomposition, Explosions, Fires, Spontaneous Combustion, Particulates from Wind Erosion
Atmosphere
Rainout Fallout
Rainout Fallout
Wind, Rainout, Fallout, Inhalation
Evaporation
Particulates and Water Solubles from Direct Contact of Waste with Surface Runoff, Surface Flows
Surface Waters
Sedimentation
Sediments
Drinking Water
Humans
Uptake by Biota
Aquatic and Terrestrial Food Chain
Particulates and Water Solubles Direct Contact of Waste with Surface Runoff, Leachate Surface Seeps
Soils – Land Surface
Uptake by Plants, Animals
Discharge to Surface Waters
Domestic Drinking Water
Groundwater
Leachate
Land-Disposed Wastes
Groundwater into Wells, Sewers
Subsurface Construction
Inhalation of Gases, Vapors
Hazard of Explosion
Lateral Migration of Gases, Vapors
Subsurface Formations
Gases, Vapors into Basements
Major Water Supply Aquifers

This handbook deals with remedial actions only on the first three of these levels. The selection of remedial actions is highly site-specific, and is discussed in detail in the next section.

2.3 SELECTION OF REMEDIAL ACTIONS

The most appropriate remedial action(s) for a problem waste disposal site can be selected only after a thorough evaluation of the problem, site characteristics and the available remedial action options. The objective of this section is to present the basic information needed to make a preliminary selection from the available options. At that point, technical personnel can use the detailed information found in Chapters 3-9 to further determine the technical and economic feasibility of potential alternatives.

The remedial action selection process can be described in the functional steps shown in Figure 2-4. These steps are discussed in detail below.

(1) Evaluate nature and extent of contamination. Prior to the selection of any remedial program for an existing site, it is necessary to have a general indication of the extent of pollution. Information must be compiled on groundwater and surface water contamination, as well as soil and biota contamination. Monitoring of these various media will provide a more detailed view of the nature and extent of pollution.

Groundwater sampling can take place from several sources, including:

- Wells drilled for a specific study
- Existing wells
- Springs or seeps

If wells in the area of the site are used as a source of potable water, they should be sampled to detect any pollution and to determine if a health emergency may exist. If no wells exist in the area, wells will have to be drilled and sampled. Any surface, groundwater, or leachate seep should also be sampled. A complete discussion of groundwater monitoring and sampling can be found in Appendix A.

Surface water and soil samples should also be taken. The location and number of samples will depend upon the size of the site, and the amount of dumping that has taken place (if this is known).

(2) Collect site-specific data. Since the selection of remedial actions is highly site-specific, site characteristics are important criteria for selection of the best remedial approaches. It is best to collect as much site-specific information as early as possible in the selection process. This step may be performed concurrently with Step 3 in the flow chart (Figure 2-4). In addition to providing information on specific media, the data should include general site characteristics that may indirectly affect the choice of remedial measures. Characteristics include quantity and quality of waste material, characteristics of site cover, the climate of the area, subsurface geology, proximity to various receptors, existing land use, and others. Table 2-1 lists important site characteristics and the general considerations they impose on the selection of remedial actions.

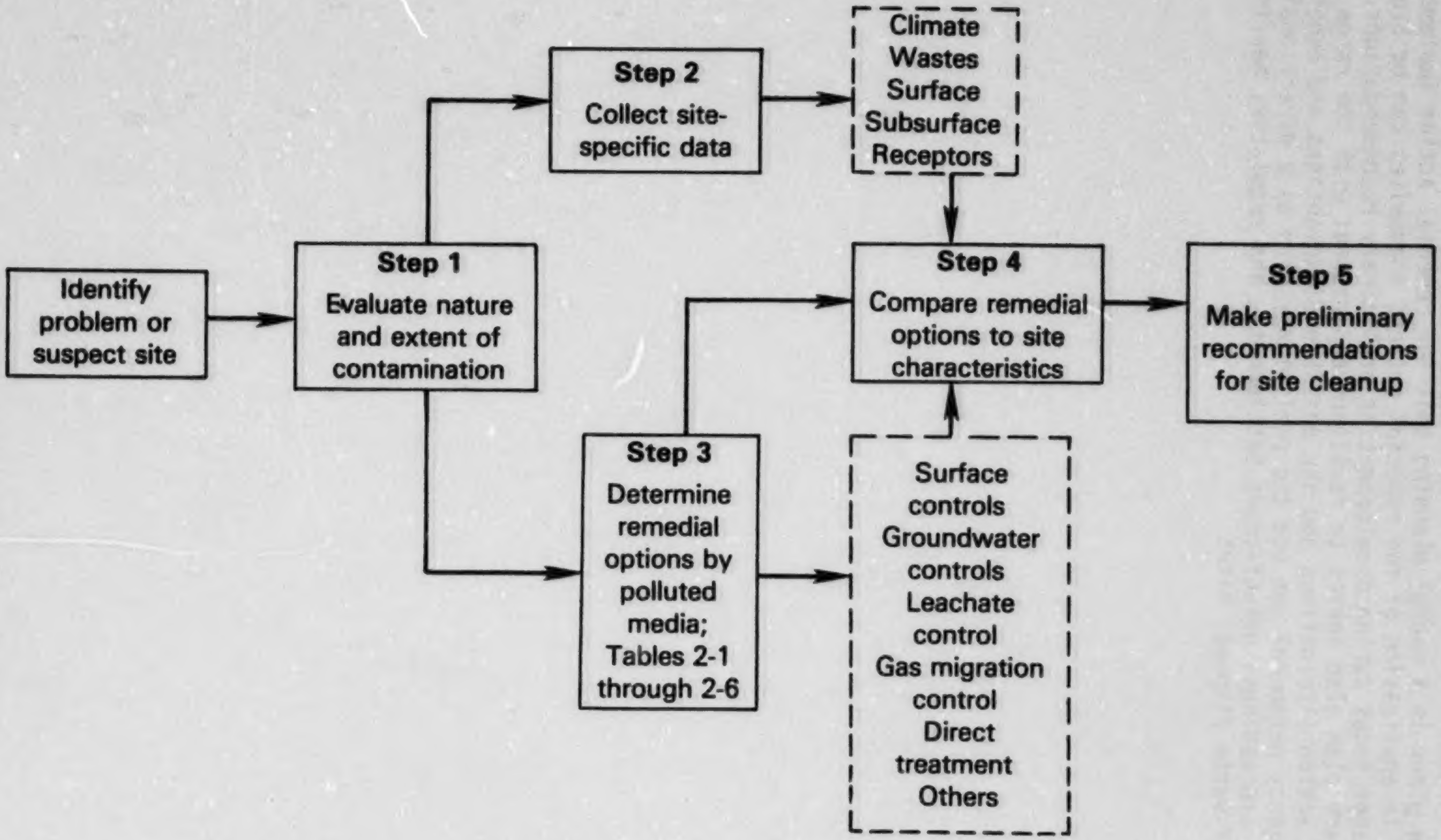
(3) Determine remedial options. For each environmental medium that is contaminated, a set of remedial actions exists that may be applicable to the site. These remedial options are summarized in Tables 2-2 through 2-7 and discussed in detail in Section 3.9. Remedial action techniques have thus been classified according to the contaminated environmental media to be remedied or controlled--surface water, groundwater, air/soil pores, soil/sediments, waste materials, and contaminated sewer and water pipelines. The individual techniques presented in each table are those that directly or indirectly treat the contaminated medium, or that control the extent of off-site environmental contamination by controlling natural transport or migration of the medium. Each remedial action technique may be used to control contamination of several different environmental media, and therefore several of the individual techniques appear in more than one table. Each table presents all available techniques applicable for the control of the indicated medium, briefly describes their medium-specific functions, indicates general site-specific applications or restrictions for the techniques, and shows where the described technique can be found in the text.

(4) Compare remedial options to site characteristics and select best remedial actions. Tables 2-1 through 2-7 can be used to compare the remedial options listed under each medium as they apply to, or are restricted by, the site characteristics information collected in Step 2. Section 2.4 illustrates the process used to compare site characteristics with remedial action alternatives. Each technique can be rated and ranked in relative order on the basis of engineering feasibility, effectiveness, and cost. In this step, certain remedial actions will be excluded because of infeasibility, while others will be singled out as most appropriate. This process should result in selection of one or two remedial measures best suited for application to each specific medium. At this point, a preliminary remedial action plan for site clean-up (Step 5) can be made.

(5) Make preliminary recommendations for site clean-up. This general model is applicable only for preparing a preliminary remedial action plan. Because of the substantial environmental impact of polluting waste disposal sites and the large costs for site clean-up, implementability, and other factors, detailed analysis of each remedial measure will be necessary before final recommendations can be made. However, the initial screening of

FIGURE 2-4

FLOW CHART FOR SELECTION OF REMEDIAL ACTIONS


Identify problem or suspect site
Step 1
Evaluate nature and extent of contamination
Step 2
Collect site-specific data
Climate
Wastes
Surface
Subsurface
Receptors
Step 3
Determine remedial options by polluted media; Tables 2-1 through 2-6
Surface controls
Groundwater controls
Leachate control
Gas migration control
Direct treatment
Others
Step 4
Compare remedial options to site characteristics
Step 5
Make preliminary recommendations for site cleanup

alternate plans is a useful planning tool for remedial action implementation. Problems in application of the remedial actions themselves can be brought into greater focus during development of preliminary recommendations. The preliminary plan also serves to familiarize personnel with the range of remedial action alternatives and the possible disadvantages and benefits from each. Agency personnel can use the preliminary plan as a directional tool in legal proceedings, out-of-court settlements, and regulatory hearings with owners of waste disposal sites.

TABLE 2-1

IMPORTANT SITE CHARACTERISTICS AND CONSIDERATIONS AFFECTING SELECTION OF REMEDIAL MEASURES

Site characteristics	Considerations
● Waste characteristics	
-- Quantity	Determines volume, size of area, affects costs
-- Chemical make-up	Determines transport paths, materials of construction
-- Toxicity	High toxicity calls for immediate action, worker safety
-- Persistence/ biodegradability	Resists decomposition/can be treated by biodegradation
-- Radioactive	Requires special materials of construction, worker safety, site security
-- Ignitable	Explosion hazards
-- Reactivity/ corrosiveness	Requires special materials of construction, potential explosion
-- Infectiousness	Calls for immediate action, worker safety
-- Solubility	Affects hydrology migration
-- Volatility	Affects migration in gaseous state
● Climate	
-- Precipitation	Humid areas - abundant surface water, shallow groundwater table Arid areas - high wind and water erosion potential, deep groundwater table
-- Temperature	Affects physical processes such as rates of reaction, volatilization, sealed container pressure as well as microbial degradation and transformation processes.

--continued--

TABLE 2-1 (Continued)

Site characteristics	Considerations
● Surface characteristics	
-- Soil texture and permeability	Coarse textured (sandy) soils have greater permeability and transmit liquid and gases faster than fine textured (clay) soils
-- Soil moisture content	Wet soils are less permeable to gases than dry soils
-- Slope	Steeper slopes have greater runoff, less infiltration
	Very steep or unbroken slopes have high erosion potential
-- Vegetation	Increases infiltration, decreases erosion
● Subsurface characteristics	
-- Depths to groundwater	Deep - higher pumping costs
	Shallow - may require lowering water table
-- Permeability	Permeable soils readily transmit water and gases
	Low permeability causes difficulty in pumping; drainage
-- Depths to bedrock	Shallow impermeable bedrock may cause leachate surface seepage; shallow or deep permeable bedrock may cause rapid and extensive contaminant migration
	Deep - limit on trench excavation depth
-- Direction of groundwater flow and points of discharge	Direction of flow toward point of use presents a significantly adverse impact; point(s) of discharge must be known to assess areal extent of contamination and degree of impact.

--continued--

TABLE 2-1 (Continued)

Site characteristics	Considerations
● Receptors	Nearby working and residential populations, farms, orchards, grazing lands, natural areas, critical habitats may require immediate relief
● Existing land use	Maintenance of site security, protection of equipment, and soil cover from accidental abuse; vandalism

2.4 EXAMPLE SITE

The selection of appropriate remedial actions is a complex process, involving a detailed consideration of site and waste characteristics and a feasibility study on the applicability of various remedial actions for the specific site. This section presents an example site and a discussion of the rationale used in determining what remedial actions are feasible from an engineering viewpoint. It is important to realize that this preliminary selection process would be followed by a detailed design feasibility and cost analysis, based on information in Chapters 3-9, as well as the institutional, legal, and political considerations before final selection of remedial actions.

The example used describes a hypothetical disposal site in terms of site dimensions, type of waste, climatology, topography, geology, hydrology, and closeness to receptors. All remedial action options for dealing with the specific site problems are addressed. Each option is considered from the standpoint of its compatibility with wastes and site characteristics, and its relative ability to remedy the problem. This information is summarized in Figure 2-5.

TABLE 2-2

SUMMARY OF AVAILABLE REMEDIAL ACTION TECHNIQUES FOR CONTAMINATED SURFACE FLOWS

Technique	Section number	Functions	Applications/restrictions
● Surface sealing/capping	3.1	Isolates waste from contact with surface runoff and infiltration; stabilizes surface of site, controls off-site transport of contaminated sediments and debris; prevents surface leaks of leachate; supports revegetation	All land disposal sites; most effective when combined with grading and revegetation; requires suitable capping and cover materials
● Grading	3.2	Shapes surface topography to provide for non-erosive runoff and and minimize infiltration; supports revegetation	All land disposal sites; most effective when combined with surface sealing with revegetation; may require special landfill equipment
● Revegetation	3:3	Stabilizes site surface; controls erosion by wind and water; controls off-site transport of contaminated debris; enhances surface sealing; may prepare site for future re-use	All land disposal sites; only recommended for properly sealed sites; may require irrigation in arid climates; most effective when combined with grading; may require special construction techniques and long-term maintenance
● Surface water diversion and collection structures	3.4	Upslope or at perimeter of site, channels runoff around critical areas; downslope or onsite, controls off-site erosive	All land disposal sites in sloping areas, surface impoundments; most suited for wet climates; often provides only short-term control

--continued--

TABLE 2-2 (Continued)

Technique	Section number	Functions	Applications/restrictions
- Dikes and berms - Ditches, diversions and waterways - Terraces and benches - Chutes and downpipes - Levees		transport of contaminated sediments; collects/channels contaminated runoff to basins/traps	for samll drainage areas; associated maintenance costs; most effective when combined with grading and revegetation
● Seepage basins	3.4.6	Collects surface runoff from diversion structures and provides for recharge to groundwater	Wherever diversion structures have been implemented and where soil permeability is not too low to allow for recharge
● Sedimentation basins/traps - check dams - sedimentation basins/ponds	3.4.7	Collects and detains contaminated sediments eroded from disposal site surface; sediment-laden surface runoff intercepted and channeled to these structures; prevents contamination of local water-courses by disposal site	All land disposal sites with sediment erosion problems; must be located in fairly remote areas for large sediment pond construction; smaller sediment traps for basins only effective for small drainage areas
● Leachate control - collection - recirculation - treatment	5.0 App. B	Controls off-site migration of surface leachate seeps (e.g., at base of fill) by collecting and treating or recirculating leachate	All disposal sites with surface seepage of leachate; particularly applicable to sites located on bedrock, where shallow groundwater exists, or with impermeable sub-layer

--continued--

TABLE 2-2 (Continued)

Technique	Section number	Functions	Applications/restrictions
● Treatment of contaminated surface waters	5.4 App. B	Removes contaminants by physical, chemical, and/or biological treatment methods	For contaminated surface runoff or natural water-courses that must be treated

TABLE 2-3

SUMMARY OF AVAILABLE REMEDIAL ACTION TECHNIQUES FOR CONTAMINATED GROUNDWATER

Technique	Section number	Functions	Applications/restrictions
● Surface sealing	3.1	Indirectly controls groundwater contamination by reducing surface water infiltration (provides impermeable barrier), thereby minimizing leachate generation	All land disposal sites; most effective when combined with grading and revegetation; requires suitable capping and cover materials
● Grading	3.2	Indirectly controls groundwater contamination by promoting surface runoff and reducing infiltration, therefore minimizing leachate generation	All land disposal sites; most effective when combined with surface sealing and revegetation; may require special landfill equipment
● Revegetation	3.3	May be used to dry surface layers of filled refuse through root uptake/evapotranspiration, reducing volume of leachate generated, and thereby indirectly controlling groundwater contamination	This function of revegetation may be offset by enhanced detention infiltration of surface runoff; site-tolerant species will be effective; may be effective at landfill sites with poorly drained surface layers and non-phytotoxic wastes near the surface
● Surface water diversion and collection structures - Dikes and berms - Ditches, diversions	3.4	Upslope of sites may indirectly control groundwater contamination by intercepting and diverting surface runoff around site, reducing opportunity for runoff infiltration and minimizing leachate generation	Structures must be upslope or at perimeter of disposal site to isolate site surface from contact with storm runoff; most suited for wet climates; often provide only short-term control for small drainage areas; associated maintenance

--continued--

TABLE 2-3 (Continued)

Technique	Section number	Functions	Applications/restrictions
and waterways - Terraces and benches - Chutes and downpipes			costs; most effective when combined with grading and revegetation
● Impermeable barriers - grout curtain - slurry wall - sheet piling	4.1	Upgradient from or around sites, diverts uncontaminated groundwater flow away from wastes; downslope or around sites contains/collects contaminated groundwater to limit extent of aquifer pollution or protect off-site wells	All land disposal sites and surface impoundments with groundwater contamination; requires expensive preconstruction geotechnical evaluation, limited bedrock depths of under 60 feet. Compatibility of wastes with grouts and, to a lesser extent, slurry walls has not been fully tested. Grouts not suitable to poorly permeable soils
● Permeable treatment beds	4.2	Adsorption, precipitation, or neutralization of certain groundwater contaminants downgradient of polluting disposal sites	Applicable to any land disposal site or surface impoundment with contaminated groundwater flowing downgradient of site; carbon adsorption very costly; not a proven technique
● Groundwater pumping	4.3	Lowers water table to prevent groundwater contact with buried or impounded wastes; lowers water table to prevent surface discharge or contaminated groundwater; contains or collects a leachate plume for delivery to treatment system	Land disposal sites and surface impoundments that are contaminating local aquifers; particularly useful when dealing with permeable bedrock, where impermeable barriers cannot contain vertical migration

--continued--

TABLE 2-3 (Continued)

Technique	Section number	Functions	Applications/restrictions
● Bioreclamation	4.4	Bacterial degradation/removal of petrochemical contaminants and other organics as groundwater is recycled between pump stations	Not effective for groundwater contaminated by heavy metals, certain chlorinated organics, or other nonbiodegradables; short-term treatment only; may be very costly
● Leachate control - collection - recirculation - treatment	5.0 App. B	Intercepts subsurface leachate before it migrates to groundwater; collects and transports leachate to retreatment system or for recirculation	All disposal sites (landfills, surface impoundments) with subsurface leachate generation; limited applicability where soils are of low permeability; may not intercept all leachate if site is very large

TABLE 2-4

SUMMARY OF AVAILABLE REMEDIAL ACTION TECHNIQUES FOR CONTAMINATED AIR/SOIL PORE SPACES

Technique	Section number	Functions	Applications/restrictions
● Surface sealing	3.1	Horizontal sealing provides impermeable barrier to upward	All land disposal sites
● Gas barriers	6.4	migration/surface escape of decomposition gases and volatiles; vertical sealing prevents lateral movement; layered sealing systems may channel gases to vents and treatment structures	Layered systems most effective for control of gas migration Vertical barriers should not be used alone for control of lateral migration; clay may crack in arid regions
● Gas ventilation systems - pipe vents - trench vents	6.2, 6.3	Prevents lateral subsurface migration of gases; safely vents hazardous gases to the atmosphere or to treatment structures	Applicable as a remedial technique for control of volatile toxics or methane and decomposition gases at land disposal sites
● Gas collection and treatment	6.4, 6.5	For control of volatile toxics, and malodorous decomposition gases, removal or destruction of pollutants by thermal oxidation or adsorption	All land disposal sites; generally cost-effective

TABLE 2-5

SUMMARY OF AVAILABLE REMEDIAL ACTION TECHNIQUES FOR CONTAMINATED SOIL AND SEDIMENTS

Technique	Section number	Functions	Applications/restrictions
● Surface sealing	3.1	Controls off-site transport of contaminated surface soil by capping waste site and stabilizing cover soil; prevents leachate seeps and subsequent contamination	All land disposal sites; most applicable to wet climates
● Grading and revegetation	3.2, 3.3	Controls off-site erosion of cover soil; binds soil particles, protects from wind and rain	All land disposal sites; most effective when combined with surface sealing; may be costly in arid climates
● Surface water diversion and collection - diversions - benches/ terraces - sediment traps/ basins	3.4	Upslope of sites, diverts eroding runoff; downslope or on site surface, slows runoff, controls soil erosion, channels sediment-laden runoff to collection structures (traps/basins) or stabilization outlets; traps and collects sediments	All land disposal sites; structures often temporary in nature; for small drainage areas; usually combined with revegetation and grading for long-term erosion control
● Leachate control	5.0 App. B	Indirectly functions to prevent soil contamination by collecting and treating leachate that might otherwise migrate off-site	All land disposal sites and surface impoundments; effectiveness limited in poorly permeable soils

--continued--

TABLE 2-5 (Continued)

Technique	Section number	Functions	Applications/restrictions
● Disposal of dredged sediments	9.3 7.3 7.4	Safe disposal of contaminated sediments in secure landfill or by incineration	Only cost-effective for large volumes of dredged/excavated sediments
● Wet excavation techniques - stream diversion - mechanical excavation - hydraulic dredging - dewatering	9.1, 9.2	Removes contaminated sediments from streams, rivers, and wetlands that may be ecologically fragile or important as public water sources	For contaminated sediments that have been eroded from the site and deposited in stream beds or wetlands; only cost-effective for removing large volumes of sediments; mechanical excavation is more cost-effective for small, low flow streams; may not be feasible for very remote, inaccessible sites

TABLE 2-6

SUMMARY OF AVAILABLE REMEDIAL ACTION TECHNIQUES FOR HAZARDOUS WASTES

Technique	Section number	Functions	Applications/restrictions
● Mechanical excavation	7.1	Removes waste materials from site for treatment or secured disposal by power shovel, clamshell, etc.	Landfills, small surface impoundments with high-solids waste material
● Hydraulic dredging	7.2	Pumps waste materials to treatment, or for transportation to secured disposal	Surface impoundments with pumpable solids
● Land disposal	7.3	Disposes of waste materials in impoundments, landfills, and landforms	Most widely used method for waste disposal; improper disposal can result in air pollution, groundwater and surface water contamination; RCRA requirements will markedly increase the cost but will provide for more sound disposal methods
● Incineration	7.4	Thermally oxidizes waste material in controlled environment.	Most effective for all organic wastes, especially those with low flash points and containing relatively low ash contents. Applicable to wastes that are oxidizable at temperatures below 2500°F

--continued--

TABLE 2-6 (Continued)

Technique	Section number	Functions	Applications/restrictions
● Wet-air oxidation	7.5 App. B	Oxidizes waste material by low-temperature thermal air	Most economical for wastes with high COD; may be used in conjunction with biological treatment
● Solidification	7.6	Incorporates waste material into immobile matrix such as cement or resin	Most economical for small quantities of waste. Waste material must be compatible with solidification agent. Not well demonstrated for nonradioactive wastes; may leach from some matrices over time
● Encapsulation	7.7	Surrounds waste material with impermeable coating	Most applicable to containerized waste materials or dewatered sludges; not fully demonstrated; costly
● Solution mining technical	7.8.1	Treats the wastes in-place by mobilizing contaminants or flushing them through to groundwater, and collecting	Most applicable to surface impoundments; may eliminate need for hazardous excavation; best suited for flushing heavy metals and basic organics; difficult to determine extent to which solution makes contact with wastes; generally used with groundwater pumping or leachate collection; not well demonstrated
● In-situ solidification	7.8.2	Injects waste solidification agents directly into waste site	Applicable to liquid wastes from surface impoundments, well defined landfill sections. Not applicable to containerized wastes

--continued--

TABLE 2-6 (Continued)

Technique	Section number	Functions	Applications/restrictions
● In-situ neutralization/detoxification	7.8.2	Neutralizes or immobilizes wastes by application of a neutralization agent such as lime to the waste material	Most applicable to surface impoundments, disposal sites with permeable surfaces; metal-bearing wastes. Degree of effectiveness difficult to determine
● Microbial seeding	7.8.3	Biodegrades organic wastes	Most effective for landforms and surface impoundments; can degrade a wide range of organics when acclimated; degradation process is slow and requires adequate aeration

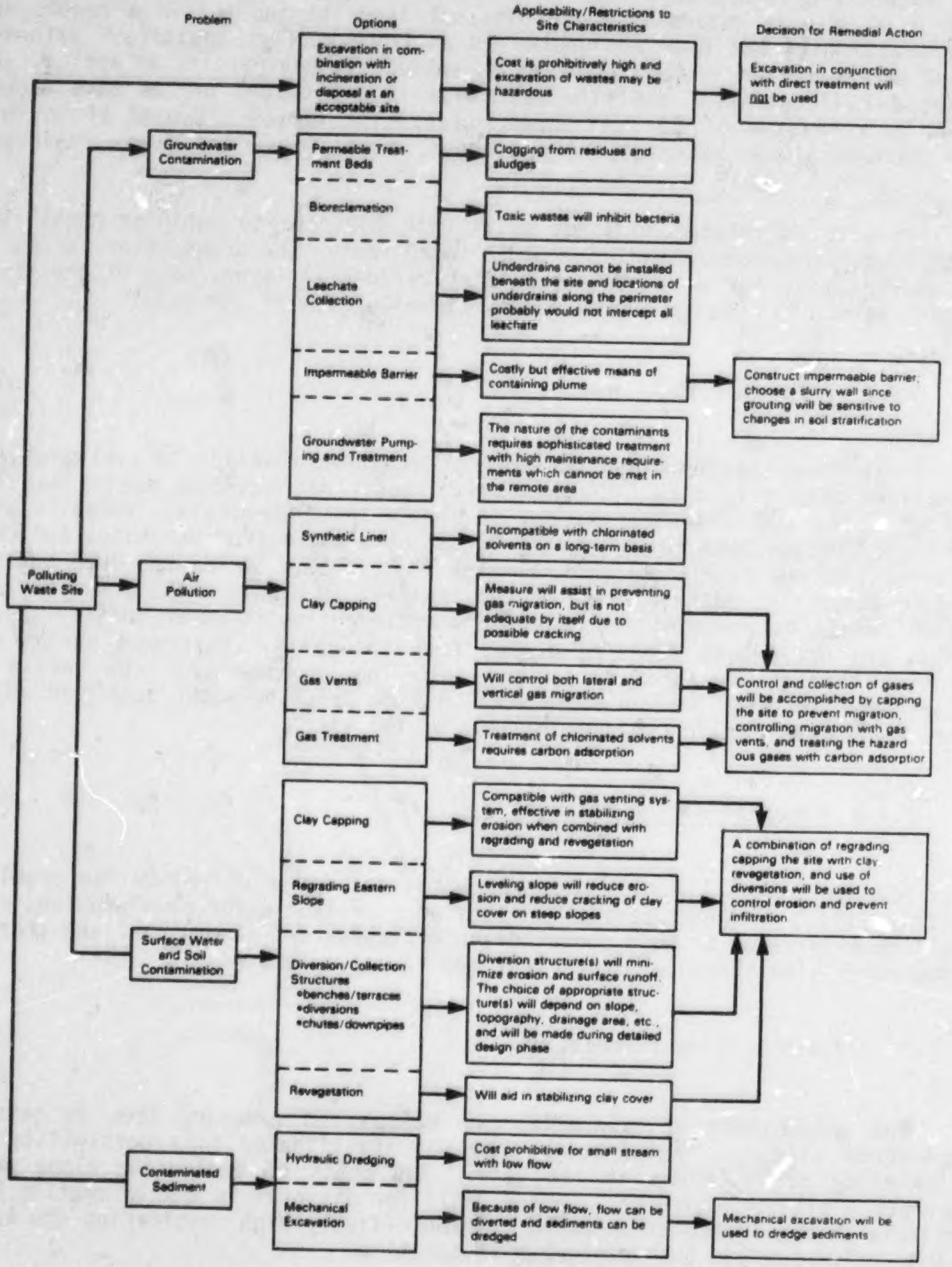
TABLE 2-7

SUMMARY OF AVAILABLE REMEDIAL ACTION TECHNIQUES FOR CONTAMINATED WATER AND SEWER LINES

Technique	Section number	Functions	Applications/restrictions
● In-situ cleaning - scouring - flushing - dredging - suction cleaning	8.1.1.	Cleans interiors of municipal sewer and water pipelines infiltrated by contaminated sediments or groundwater; removes infiltrated contaminants	Most applicable to contaminated gravity sewer lines; most techniques well established and cost-effective
● Leak detection and repair - pipeline inspection - grouting - relining	8.1.2	Allows discovery and repair of leaks, cracks etc. (points of infiltration/exfiltration).	Most applicable to contaminated sewer lines; techniques well established and generally cost-effective
● Pipeline removal and replacement	8.1.3	Replaces badly damaged sewer lines or contaminated water mains	May be only option feasible for contaminated public water mains; very costly for deep pipelines

FIGURE 2-5

REMEDIAL ACTION DECISION FLOW FOR HYPOTHETICAL WASTE DISPOSAL SITE


Problem
Options
Applicability/Restrictions to Site Characteristics
Decision for Remedial Action
Polluting Waste Site
Excavation in combination with incineration or disposal at an acceptable site
Cost is prohibitively high and excavation of wastes may be hazardous
Excavation in conjunction with direct treatment will not be used
Groundwater Contamination
Permeable Treatment Beds
Clogging from residues and sludges
Bioreclamation
Toxic wastes will inhibit bacteria
Leachate Collection
Underdrains cannot be installed beneath the site and locations of underdrains along the perimeter probably would not intercept all leachate
Impermeable Barrier
Costly but effective means of containing plume
Construct impermeable barrier; choose a slurry wall since grouting will be sensitive to changes in soil stratification
Groundwater Pumping and Treatment
The nature of the contaminants requires sophisticated treatment with high maintenance requirements which cannot be met in the remote area
Air Pollution
Synthetic Liner
Incompatible with chlorinated solvents on a long-term basis
Clay Capping
Measure will assist in preventing gas migration, but is not adequate by itself due to possible cracking
Gas Vents
Will control both lateral and vertical gas migration
Gas Treatment
Treatment of chlorinated solvents requires carbon adsorption
Control and collection of gases will be accomplished by capping the site to prevent migration, controlling migration with gas vents, and treating the hazardous gases with carbon adsorptior
Surface Water and Soil Contamination
Clay Capping
Compatible with gas venting system, effective in stabilizing erosion when combined with regrading and revegetation
Regrading Eastern Slope
Leveling slope will reduce erosion and reduce cracking of clay cover on steep slopes
A combination of regrading capping the site with clay revegetation, and use of diversions will be used to control erosion and prevent infiltration
Diversion/Collection Structures
•benches/terraces
•diversions
•chutes/downpipes
Diversion structure(s) will minimize erosion and surface runoff. The choice of appropriate structure(s) will depend on slope, topography, drainage area, etc., and will be made during detailed design phase
Revegetation
Will aid in stabilizing clay cover
Contaminated Sediment
Hydraulic Dredging
Cost prohibitive for small stream with low flow
Mechanical Excavation
Because of low flow, flow can be diverted and sediments can be dredged
Mechanical excavation will be used to dredge sediments

2.4.1 Site Description

Figure 2-6 illustrates the characteristics of the hypothetical site. The site is a 20-acre hazardous waste disposal landfill located in a remote and arid area, which has been designated as environmentally sensitive. Although annual precipitation is low, the area is subject to heavy rains in spring. As Figure 2-6 illustrates, the site was begun in a pit, and wastes have accumulated to a maximum of 20 feet above grade. The wastes disposed of in this site include waste solvents, oily sludges, pesticides, and gummy residues.

The site is located in sandy soils with moderate to rapid permeability. Depth to the groundwater varies from 12 to 20 feet. The groundwater serves as the water supply for a sparsely populated residential area, east of the site. A small stream is located approximately 1/4 mile east of the site.

2.4.2 Problem

Field investigations and monitoring data have revealed several problems associated with this site. Uncontained disposal of hazardous wastes has resulted in the environmental release of highly toxic chemicals. Volatile solvents and hydrocarbons are migrating to the atmosphere from the site, and loss of vegetation in a critical natural area east of the stream has been attributed to this. The eastern slope of the landfill is highly eroded, and contaminated runoff is reaching the stream and contributing to water quality degradation and occasional flooding during peak rainfalls. Hazardous wastes are leaching through the sandy soil and into the groundwater. Low levels of chlorinated organics have been detected in the drinking water supply of residents located hydraulically downgradient of the site.

2.4.3 Remedial Action Measures

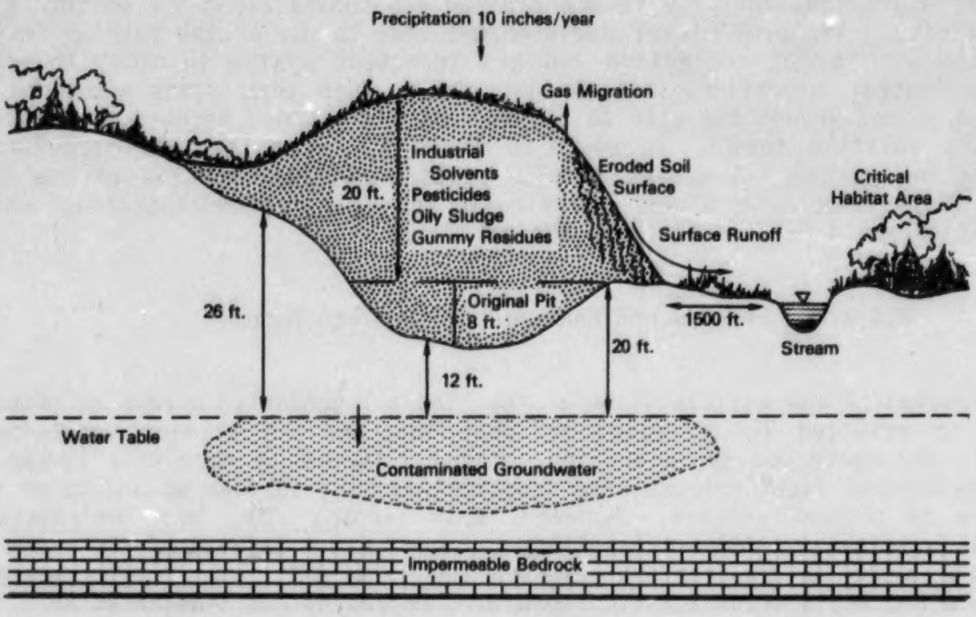
Several remedial action measures were considered to remedy the problems at this site. Although several options were available for each problem, many could be eliminated through consultation of Tables 2-1 through 2-7 and considering other site characteristics that may affect selection.

2.4.3.1 Direct Treatment

The possibility of excavating the wastes and removing them to better-engineered sites or incinerating them was investigated as a possibility for alleviating all problems at the site. The costs of excavation alone would approach a million dollars. Added costs for disposal in a site meeting RCRA criteria or for incineration would be prohibitively high considering the large volume of wastes and the remoteness of the site.

FIGURE 2-6

CHARACTERISTICS OF SITE I


Precipitation 10 inches/year
Gas Migration
Industrial Solvents
Pesticides
Oily Sludge
Gummy Residues
20 ft.
Eroded Soil Surface
Critical Habitat Area
Surface Runoff
Original Pit
8 ft.
26 ft.
1500 ft.
20 ft.
Stream
12 ft.
Water Table
Contaminated Groundwater
Impermeable Bedrock

2.4.3.2 Air Pollution from Vapor Migration

Monitoring data indicated that volatile chlorinated solvents and aromatic hydrocarbons are migrating vertically from the surface of the site. First, the possibility of controlling gas migration through use of synthetic liners was considered; however, no synthetic materials are known to be of certain compatibility for long-term use with the types of wastes present, particularly chlorinated solvents. Use of a clay cover would afford some containment of gas, but the cover would be subject to cracking in the arid climate of this site. The potential for cracking could be reduced by using currently marketed reinforced agents, and the clay cover seemed attractive from the point of view that it would substantially reduce erosion and runoff along the eastern slope of the site. The preliminary decision was made to use a clay barrier in combination with a gas collection and gas treatment system in order to effectively control migration of hazardous gases. Gas pipe vents would be installed in and around the site to control gas migration. Because the landfill contains volatile toxics, it would be necessary to install a closed forced-ventilation system. A series of pipe vents would be connected to the manifold, which leads to a blower and finally to a gas treatment system. Carbon adsorption would be used to treat the gas.

2.4.3.3. Erosion and Contaminated Surface Runoff

Erosion of the eastern slope of the landfill presented a complex problem, since it resulted in increased sediment load to the stream, contaminated runoff, and increased gas migration. Since it would be necessary to cap the site to control vapor release, the decision was made to take advantage of this measure to control erosion. However, clay capping alone was inadequate to control erosion and prevent runoff into the stream. Because of the steep and variable slope on the eastern side of the landfill, clay capping would be unstable and would erode again. Therefore, regrading was considered necessary if the clay capping were to be effective over the long term. However, large quantities of hazardous wastes are located close to the surface, and the amount of regrading that could be done was limited.

To further limit the potential for erosion, two additional control measures were considered: revegetation of the slope, and the use of diversion and/or collection devices. With an effective gas venting system, it would be possible to revegetate the slope without incurring phytotoxic effects from waste gases. Therefore, revegetation with native species was decided upon.

Several alternatives were considered for diversion and/or collection of surface waters to minimize or eliminate erosive flows of water during peak rain periods. Alternatives for consideration included bench terraces, diversion ditches, and downpipes.

Selection of specific surface flow controls must take into account topography, soil type, drainage area, etc., which must be further analyzed. Unfortunately, the preliminary selection criteria for diversion structures cannot provide further specification.

2.4.3.4 Contaminated Sediments

Contaminated sediments in the stream adjacent to the landfill have been identified as a problem because the stream is situated in an environmentally sensitive area. It was recommended to dredge the sediments within the vicinity of the site. Available options for accomplishing this objective include mechanical excavation or hydraulic dredging. Because the stream has a low volume of flow throughout most of the year, the decision was made to divert the flow and use mechanical excavation rather than hydraulic dredging.

2.4.3.5 Groundwater Contamination

As Figure 2-5 indicates, the options considered for alleviating the groundwater pollution problem include:

- Impermeable barriers
- Groundwater pumping
- Permeable treatment beds
- Bioreclamation
- Leachate collection

Bioreclamation and permeable treatment beds were readily eliminated as feasible alternatives because of the nature of the hazardous wastes. Gummy residues and oily sludges would clog a permeable treatment bed, thereby minimizing its effectiveness. Bioreclamation would be poorly suited to handling the toxic mixture of wastes from this site, since the wastes would inhibit microbial proliferation. The decision to use leachate collection, impermeable barriers, or groundwater pumping required considerably more investigation of site characteristics.

Because of the large size of the site, and the infeasibility of safely installing underdrains beneath the site, it was very unlikely that a leachate collection system would be effective; underdrains located along the site perimeters probably could not intercept all the leachate beneath the site. This point was verified during the detailed design phase.

A depth of water table of 12 to 26 feet below the site, the presence of impermeable bedrock at a depth of 32 to 38 feet along the site perimeter, and the presence of sandy soils with only occasional clay lenses suggested that either groundwater pumping or impermeable barriers would be suited to containing groundwater contamination. The recommended final decision to use impermeable barriers was based on two important considerations. Monitoring data indicated that the groundwater was highly contaminated and treatment of the waste, though feasible, would be very costly. The second consideration was that, because of the nature of the wastes and contaminated groundwater, the treatment system would require a highly qualified operator and daily maintenance staff. However, because of the site's remote location, this requirement could not be met realistically, and the groundwater pumping and treatment was judged infeasible.

3.0 SURFACE CONTROLS

Surface controls are those remedial techniques designed to reduce surface water infiltration and to control runoff at waste disposal sites. They also serve to reduce erosion and to stabilize the surface of covered landfills. These controls minimize hazardous leachate generation and reduce off-site erosive transport of cover materials and exposed refuse. Surface control measures include capping, grading, revegetation, and runoff diversion/collection. They are often performed in conjunction with site closure and in preparation for potential reuse of the area for non-disposal activities, and are generally well-established techniques.

3.1 SURFACE SEALING

Surface sealing, or capping, is the process by which waste disposal sites are covered to prevent surface water infiltration, control erosion, and isolate and contain contaminated wastes and volatiles. A variety of impermeable cover materials and sealing techniques are available for such purposes. The choice of sealing material and method of application is dictated by site-specific factors such as local availability and costs of cover materials, desired functions of cover materials, the nature of the wastes being covered, local climate and hydrogeology, and projected future use of the site in question.

3.1.1 Description and Applications

Cover soils are spread over waste layers at most operating landfills on a daily or intermediate basis as prescribed by state and local standards in order to control vectors, odors, and windblown rubbish. These soils are generally supplied from on-site excavated fill and are not selected for special qualities. Soils used for final cover on completed fills or for capping uncontrolled waste sites, however, must be relatively impermeable (low permeability coefficient, k) and erosion-resistant. Fine-grained soils such as clays and silty clays have low k values and are therefore best suited for capping purposes because they resist infiltration and percolation of water. These fine-grained soils, however, tend to be easily eroded by wind (Table 3-1) especially in arid climates, where coarse, heavy-grained gravels and sands provide more suitable cover (Lutton et al., 1979).

TABLE 3-1

.KING OF SOIL TYPES BASED ON PERCOLATION CONTROL AND RESISTANCE TO WIND EROSION[1]

	Ranking for	
Soil type	Impeding percolation	Resistance to wind erosion[2]
Gravel	10	1
Silty gravel	7	3
Clayey gravel	5	5
Sand	9	2
Silty sand	8	4
Clayey sand	6	6
Silt	4	7
Silty clay	2	8
Clayey silt	3	9
Clay	1	10

---

[1] Source: Lutton, 1978; Lutton et al., 1979.
[2] Assuming low soil moisture and no cover vegetation.

Blending of different soil types broadens the grain size distribution of a soil cover and minimizes its infiltration capacity. Well-graded soils are less permeable than those with a small range of grain sizes, and mixing of local coarse and fine-grained soils is a cost-effective method of creating stronger and less porous cover soil (Lutton et al., 1979). For example, when well-graded fine soils are not available locally, the addition of gravel or sand to fine-grained silts and clays enhances strength and reduces percolation. Similarly, additions of clay to a sandy or silty cover material will lead to dramatic reductions in the k value of the soil. Blending can often be accomplished in place using a blade or harrow to turn and mix the soil to suitable depths (Lutton et al., 1979).

Chemical stabilizers and cements can be added to relatively small amounts of cover soils to create stronger and less permeable surface sealants. Portland cement or bitumen (emulsified asphalt or tar) is suitable for mixing with sandy soils to stabilize and waterproof the soils. Site-specific mixing, spreading and compacting procedures are required. For a soil-cement cover, approximately 8 percent (by weight) dry cement is blended into the soil with a rotary hoe or tiller as water is added. Intermittent sprinkling over several days may be required before compaction and solidification are achieved (Lutton et al., 1979).

Cover soils may also be treated with lime and/or fly ash, which contribute pozzolanic (cementing) properties to the resulting mixture, optimize the grain size distribution and reduce shrink/swell behavior. Lime applied as 2-8 percent (by weight) calcium oxide or hydroxide is suitable for cementing clayey soils. Sands and gravels are more suitable for cementing clayey soils. Sands and gravels are more suitable for combined limefly ash treatment then are finer-grained soils (Lutton et al., 1979). Rotary tiller mixing followed by water addition and compaction is the general application sequence for these mixtures. Also, additions of lime are recommended for neutralizing acidic cover soils, thereby reducing the leaching potential of heavy metals. If a synthetic liner is present, liner life can be prolonged by lime addition to supporting soil (Fields and Lindsay, 1975).

Other cover soil additives include chemical dispersants and swell reducers. Soluble salts such as sodium chloride, tetrasodium pyrophosphate, and sodium polyphosphate are added primarily to fine-grained soils with clay minerals to deflocculate the soils, increase their density, reduce permeability, and facilitate compaction. Additives are more effective with montmorillonite clay than with kaolinite or illite. Because soils in the northeast and midwest are usually low in montmorillonite, site-specific testing should be undertaken before using such additives with soils in these areas.

Bentonite is a natural clay, composed primarily of montmorillonite, which is extremely fine-grained and absorbent. Its high swelling properties make it suitable for mixing with soil and water to produce a low permeability cover layer for waste sites. Any clay cover layer must be kept moist to avoid cracking. This is usually accomplished by covering another soil layer in which vegetation is planted.

Portland cement, concrete, or mortar can be mixed with water and spread over well-compacted bases to cover and to seal selected disposal sites. Bituminous concrete or mortar composed of 7-10 percent asphalt is another rigid cover material that can be used effectively to pave over the surface of a landfill.

Sprayed bituminous membranes, generally one quarter-inch thick, require special equipment for application. The asphalt is blown hot with a phosphoric catalyst, and solidifies as it cools. These membranes may be reinforced by polypropylene fabric underliners. They may also be prefabricated as bitumen-coated fiber mats reinforced with jute or hemp (Lutton et al., 1979).

For chemically severe surface environments, sprayed sulfur membranes are relatively new products that can substitute for asphalt or cement disposal site covers. Polyurethane foam is another cover material with possible applicability at waste disposal sites (Lutton et al., 1979).

Flexible synthetic membranes include cover liner materials made of polyvinyl chloride (PVC), chlorinated polyethylene (CPE), ethyene propylene rubber, butyl rubber, hypalon and neoprene (synthetic rubbers), and elasticized polyolefin. Synthetic liners are generally more expensive and labor-intensive sealing materials that require special field installation methods. Thin sheets are available in sections of variable width, and the sheets are overlain and spliced in the field (according to manufacturer's specifications). Special adhesives and sealants are used to ensure liner integrity.

Another class of available cover materials includes waste materials such as industrial residues and dredged sediments. Fly ash can be used as a cover soil additive and as a cover layer itself. Essentially a silt with pozzolanic qualities, fly ash condenses as a boiler emission at coal-fired power plants and collects on electrostatic precipitators. Forty million tons are generated annually, predominantly from sources east of the Mississippi River (Figure 3-1). Fly ash is usually modified by additions of lime before incorporation into soil cover; however, it has been used directly as an intermediate cover in West Virginia landfills (Lutton et al., 1979).

Bottom ash and slag are the boiler bottom counterparts of stack fly ash. These materials are of coarser grain size than fly ash and higher k, and may be blended into soil cover to improve particle size distribution and decrease permeability (Lutton et al., 1979).

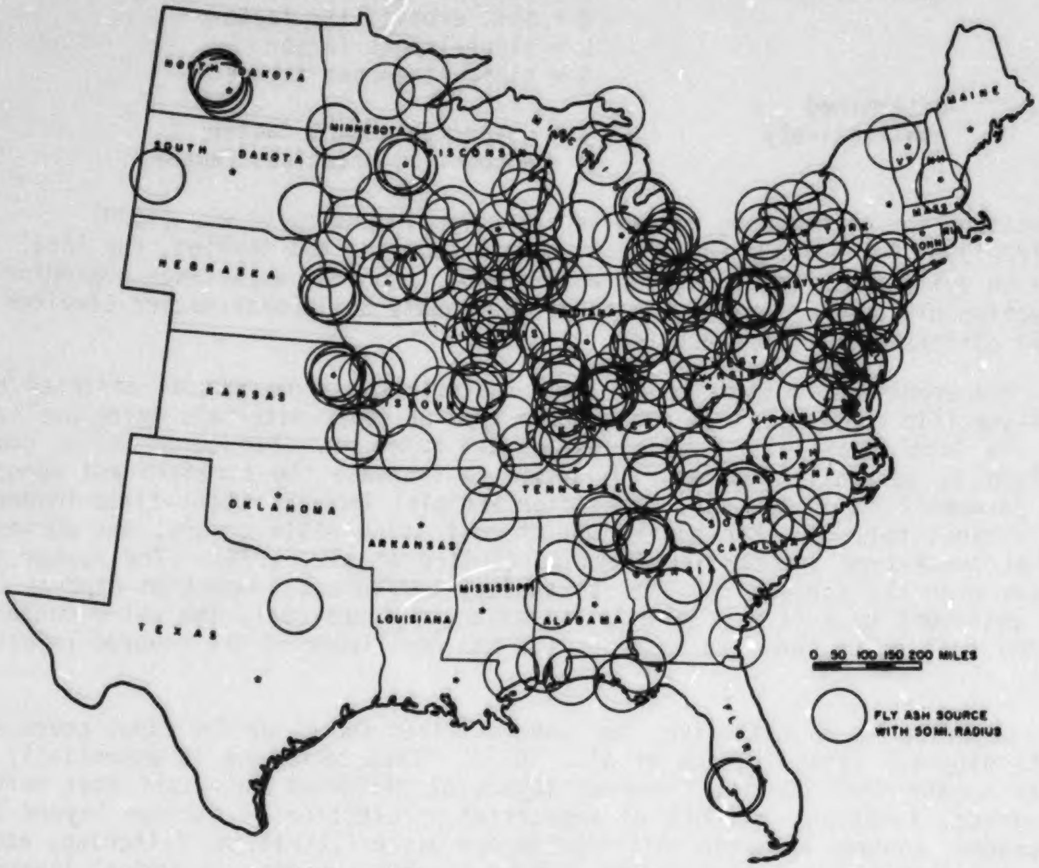
Furnace slag and incinerator residue are two additional waste materials of gravelly and sandy size that may be suitable for blending into soil cover for slope erosion protection. Rocky overburden from mines, quarries, and sand and gravel pits may also be locally useful as soil cover substitutes. Heavy applications of durable crushed stone, gravel, or clinkers (over-cooked bricks) may be used to stabilize contaminated surface soils at landfills and dumps (EPA, 1976). Nontoxic industrial sludges such as paper mill sludge, dredged materials such as reservoir and channel silt, and composted sewage sludge are other waste materials that may be applied as substitutes or supplements to conventional cover material (Lutton et al., 1979).

3.1.2 Design and Construction Considerations

The design and implementation of a cost-effective capping strategy involves first the selection of an appropriate cover material. Site-specific cover functions--control of water infiltration and gas migration, water and wind erosion control, crack resistance, settlement control and waste containment, side slope stability, support of vegetation, and suitability for future site use--may be ranked in order of importance to facilitate this selection (Lutton et al., 1979). For soils that may potentially be used in capping, laboratory and field testing of physical and chemical properties may be necessary when the choice is not clear-cut. Void ratio, porosity, water content, liquid and plastic limits, shrinkage limit, pH and nutrient levels, shear

FIGURE 3-1

FLY ASH SOURCES EAST OF ROCKY MOUNTAINS (Source: Lutton et al., 1979)


NORTH DAKOTA
SOUTH DAKOTA
MINNESOTA
WISCONSIN
NEBRASKA
KANSAS
MISSOURI
OKLAHOMA
ARKANSAS
TEXAS
LOUISIANA
MISSISSIPPI
ALABAMA
TENNESSEE
NORTH CAROLINA
SOUTH CAROLINA
FLORIDA
MAINE
VT
NH
MASS
CONN
RI
0 50 100 150 200 MILES
FLY ASH SOURCE WITH 50MI. RADIUS

resistance, compaction, permeability, shrink/swell behavior, and grain size are some of the properties that may have to be determined for competing soil types. Interpretation of data on these properties is discussed extensively in Lutton et al. (1979). Local availability and costs, of course, will influence the selection of a cover material. Costs are addressed in Section 3.1.4.

Where soil erosion control is a major consideration, the USDA Universal Soil Loss Equation (USLE) may be useful for comparing the predicted effectiveness of different cover soils. The USLE equates soil loss to the product of two quantitative terms and four qualitative variables:

$$A = R \times K \times L \times S \times C \times P$$

Where A = average soil loss in tons/acre for time period used for R

quantitative
R = rainfall and runoff erosivity index
K = soil erodibility factor
L = slope-length factor
S = slope-steepness factor

determined qualitatively
C = cover/management factor
P = supporting practices factor

Directions for determining variables are given in Lutton et al. (1979) pp 127-133. For information regarding soil sampling and testing, for local data on soils and climate, or for any form of technical assistance regarding selection of cover materials, regional and county Soil Conservation Services (SCS) offices should be consulted.

Placement and compaction of cover materials are techniques affected by site-specific considerations such as the type of cover materials being applied and the local availability of equipment and manpower. For cover soils, compaction is generally desirable in order to increase the strength and reduce the permeability of the cap. Compactor vehicles include rubber-tired loaders and various rollers. For compaction of most solid waste covers, the conventional track-type tractor is effective (Lutton et al., 1979). The number of passes over the surface required to achieve sufficient compaction depends on the equipment type (size, weight, and width of compactor), the water content of the soil cover, and the base density and resiliency of the covered refuse.

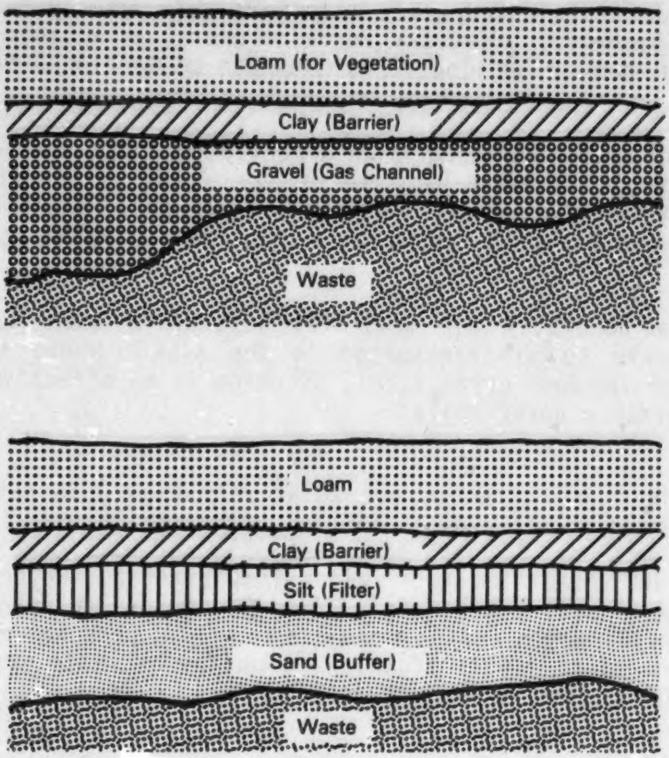
Layering is an effective, but underutilized technique for final cover at waste disposal sites (Lutton et al., 1979). This technique is essentially a cover system that combines several layers of different materials that serve integrated functions--support of vegetation, protection of barrier layers or membranes, control of water infiltration and gas exfiltration, filtering, etc. Figure 3-2 depicts examples of two layered covered systems. A typical layered cover system may be composed of the following layers:

- Topsoil - usually loose, uncompacted surface layer of loams for vegetative support; may be treated with fertilizers or conditioners
- Barrier layer or membrane - usually clayey soil with low k, or a synthetic membrane; restricts passage of water or gas
- Buffer layer - above and/or below barrier layer; protects clays from drying or cracking, synthetic membranes from punctures or tears; provides smooth, stable base; often a sandy soil
- Water/gas drainage layer or channel - poorly graded (homogeneous) sand and gravel; channels subsurface water drainage; intercepts and laterally vents gases

- Filter - intermediate grain-size layer to prevent fine particles from penetrating coarser layer; controls settlement, stabilizes cover

FIGURE 3-2

TWO TYPICAL LAYERED COVER SYSTEMS (Source: Lutton et al., 1979)


Loam (for Vegetation)
Clay (Barrier)
Gravel (Gas Channel)
Waste
Loam
Clay (Barrier)
Silt (Filter)
Sand (Buffer)
Waste

Layered cover systems are designed on a site-specific basis, depending on desired functions of the different layers and availability of materials. Some general considerations during the construction of such a final cover system are listed below (Lutton et al., 1979):

- Compact all layers except topsoil
- Cover barrier layer quickly to prevent drying

- Design for 6 to 12-inch minimum layer thickness to compensate for possible settlement or improper spreading
- Construct in small plots to allow efficient completion
- Consider seeding topsoil as it is spread

3.1.3 Advantages and Disadvantages

An evaluation of selected cover materials and cover systems must be made on a site-specific basis. However, certain general advantages and disadvantages of different surface sealing techniques can be mentioned here.

Fine-grained soils composed predominantly of clay are well-suited for final cover in humid climates because of their low permeability. However, such soils tend to shrink and crack during dry seasons. The construction of a two-layer cover system may be useful in solving such problems.

Local soils generally are much less expensive than non-native cover materials that have to be transported to the site. Where local soils are poorly graded (homogeneous grain size), blending is an effective technique for creating more suitable cover soils.

Soil additives and cements have relatively high unit costs and may require special mixing and spreading methods. Also, soils modified by additions of cement, bitumen, or fly ash-lime become rigid and more susceptible to cracking due to waste settlement or freeze-thaw stresses. Patching repairs may become necessary to seal cracks that allow for escape of volatiles and allow surface water infiltration. Also, cemented soil systems may deteriorate upon extended exposure to corrosive organic and sulfurous waste products in landfill environments (Lutton et al., 1979).

Rigid barriers such as concrete and bituminous membranes are also vulnerable to cracking and chemical deterioration, but the cracks can be exposed, cleaned, and repaired (sealed with tar) with relative ease. Concrete covers may have a design life of about 50 years, except when applied to chemically severe or physically unstable landfill environments (Lutton et al., 1979).

Plastic and rubber liners are vulnerable to tearing, sunlight, exposure, burrowing animals, and plant roots. They also require special placement and covering procedures. A maximum design life of 20 years is recommended for most synthetic membranes. Among the commercially available synthetic liners, polyethylene may be the most economical based on both performance and cost (Lutton et al., 1979). Locally generated waste materials such as fly ash, furnace slag, and incinerator residue may be inexpensive (or free) and therefore cost-effective cover materials or additives. However, such materials may leach soluble trace pollutants (e.g., sulfur, heavy metals) and may actually contribute to environmental contamination.

3.1.4 Costs

Unit costs associated with surface sealing methods for waste disposal sites are presented in Table 3-2. Most of these costs are presented as ranges, reflecting the varied sources of the cost estimates.

The costs presented in Table 3-2 do not necessarily represent national averages; they are intended to give estimates or ranges for costs of installing most of the surface sealing materials reviewed in this chapter. An example is given below to show how these unit costs are applied in the derivation of total costs for surface sealing at a hypothetical disposal site.

* * *

Assume that a cap consisting of a Hypalon surface liner surrounded by two 6-inch layers of sand and covered by eight inches of sandy loam topsoil is to be constructed over a 20-acre disposal site. Further assume that the borrow site is within 20 miles of the disposal site. The calculations involved in determining the cost for capping this site follow:

- (20 acres) (4,840 yd^2/acre) = 96,800 yd^2 ... surface area of seal
- 2 x 6-inch layers of sand = 1 ft = (1/3 yd) (96,800 yd^2) = 32,267 yd^3 ... volume of sand required
- (\$15/yd^3) (32,267 yd^3) = \$484,000 ... cost of hauling, spreading, and compaction of sand around liner
- (\$6.50/yd^2) (96,800 yd^2) = \$629,000 ... cost of Hypalon (30 mil) installation
- 8-inch layer of sandy loam = (2/9 yd) (96,800 yd^2) = 21,510 yd^3 ... volume of topsoil required
- (\$13/yd^3) (21,510 yd^3) = \$279,644 ... cost of hauling, spreading, and compaction of topsoil
- TOTAL COST = \$984,000 + \$629,200 + \$279,644 = approximately \$1.39 million

This total represents estimated capital expenditures required for seal construction only. Maintenance or repair costs will be extra.

3.2 GRADING

Grading is the general term for techniques used to reshape the surface of covered landfills in order to manage surface water infiltration and runoff while controlling erosion. The spreading and compaction steps used in grading are techniques practiced routinely at sanitary landfills. The equipment and methods used in grading are essentially the same for all landfill surfaces, but applications of grading technology will vary by site. Grading is often performed in conjunction with surface sealing practices and revegetation as part of an integrated landfill closure plan.

3.2.1 Description and Applications

Grading techniques modify the natural topography and runoff characteristics of waste sites to control infiltration and erosion. The choice of specific grading techniques for a given waste disposal site will depend on desired site-specific functions of a graded surface. A graded surface may reduce or enhance infiltration, detain or promote runoff. Erosion control may be considered a complicating variable in the design and performance of a grading scheme.

For disposal sites in wet climates (i.e., where precipitation annually exceeds evaporation and transpiration) and where subsurface hazardous leachate generation is a major problem, control of surface water infiltration is of primary importance. Manipulation of slope length and gradient is the most common grading technique used to reduce infiltration and promote surface water runoff. A slope of at least 5 percent is recommended as sufficient to promote runoff and decrease infiltration without risking excessive erosion (Lutton, 1978). More specifically, it has been suggested that the center of the disposal site be the highest elevation, with top surface slopes graded 6 to 12 percent and side slopes no steeper than 18 percent. This grading will enhance runoff and minimize infiltration and most soils will remain stable on such slopes (Tolman et al., 1978). For equal surface areas of land, doubling the slope length will increase soil losses by 1.5 times (EPA, 1976). Therefore, where off-site transport of contaminated soil due to water erosion is a major consideration, the length of graded slopes should be minimized. Minimization of slope length can be accomplished by terracing, as discussed in Section 3.4.3.

At landfill and dump sites where an effective surface sealing has been applied (e.g., clay cap or synthetic membrane and a topsoil layer), various grading techniques can be used to prepare the covered surface for revegetation. The grading methods--scarification, tracking, and contour furrowing--create a roughened and loosened soil surface that detains runoff and maximizes infiltration (EPA, 1976). Such techniques are especially important for establishing vegetation in arid regions.

TABLE 3-2

ESTIMATED UNIT COSTS FOR SURFACE SEALING METHODS AND MATERIALS

Cover material and/or method of installation	Unit costs*	Source of cost information
Topsoil (sandy loam), hauling, spreading, and grading (within 20 miles)	$\$13/yd^3$	Haseley Trucking Company (1980)
Clay hauling, spreading, and compaction	$\$8.50/yd^3$	" " " "
Sand hauling, spreading, and compaction	$\$15/yd^3$ $\$8,000-10,000$ /acre	" " " " Universal Linings, Inc. (1980)
Cement concrete (4 to 6" layer), mixed, spread, compacted on-site	$\$6-10/yd^2$	Tolman et al., 1979
Bituminous concrete (4 to 6" layer), including base layer	$\$3-5/yd^2$	Tolman et al., 1979
Lime or cement, mixed into 5" cover soil	$\$1.50-2.10/yd^2$	Tolman et al., 1979
Bentonite, material only; 2" layer, spread and compacted	$\$1.40/yd^2$	Lutton et al., 1979
Sprayed asphalt membrane (¼" layer and soil cover), installed	$\$1.50-2.50/yd^2$	Lutton et al., 1979
PVC membrane (20 mil), installed	$\$1.30-2.00/yd^2$	" " " "
Chlorinated PE membrane (20-30 mil), installed	$\$2.40-3.20/yd^2$	" " " "
Elasticized polyolefin membrane, installed	$\$2.70-3.60/yd^2$	DuPont Elastomer Chemicals Dept. (1980)

--continued--

TABLE 3-2 (Continued)

Cover material and/or method of installation	Unit costs*	Source of cost information
Hypalon membrane, (30 mil), installed	$\$6.50/\text{yd}^2$	DuPont Elastomer Chemicals Dept. (1980)
Neoprene membrane, installed	$\$5.00/\text{yd}^2$	Lutton et al., 1979
Ethylene propylene rubber membrane, installed	$\$2.70\text{-}3.50/\text{yd}^2$	" " " "
Butyl rubber membrane, installed	$\$2.70\text{-}3.80/\text{yd}^2$	" " " "
Teflon-coated fiberglas (TFE) membrane (10 mil), installed	$\$20/\text{yd}^2$	DuPont Elastomer Chemicals Dept. (1980)
Fly ash and/or sludge, spreading, grading, and rolling	$\$1.00\text{-}1.70\text{yd}^2$	Tolman et al., 1979

*Note different units for volume (yd^3) vs. surface area (yd^2) costs.

Scarification can be accomplished by harrowing along the ground contour or by dragging the bucket teeth of a front-end loader over the ground. Specially equipped crawler tractors can also perform scarification. Tracking is performed on steep slopes where movement along the contour is not feasible. A cleated crawler tractor is run up and down the slope, leaving shallow grooves that run parallel to the contour. Contour terracing or furrowing is used on long slopes in conjunction with other roughening techniques to disrupt and slow surface runoff. This effect is accomplished by running a bulldozer parallel to the contour. Dirt is allowed to dribble off the blade end, creating small depressions that interrupt downslope surface water flow (EPA, 1976).

3.2.2 Design and Construction Considerations

The design of graded slopes at waste disposal sites should balance infiltration and runoff control against possible decreases in slope stability and increases in erosion. The design of specific slope configurations, the choice of cover soil type, the degree of compaction, and the types of grading equipment used will all depend on local topography, climate, and future land use of the site.

Improperly graded slopes may deform or fail, opening cracks, exposing waste cells, and allowing lateral seepage of leachate. Soils used to cover graded slopes should be selected on the basis of shear strength and erodibility (Lutton et al., 1979). Soils high in silt and fine sand and low in clay and organic matter are generally most erodible (EPA, 1976). Also, the longer and steeper the slope is, and the sparser the vegetation cover, the more susceptible it is to erosive forces.

In grading a landfill surface before construction of a seal, two important considerations apply. First, bulky and heavy waste objects should not be filled near the surface of the site, because they may settle unevenly and deform or crack graded cover (Lutton et al., 1979). Also, to provide a firm subgrade and prevent seal failure, existing cover material should be compacted to a Proctor density of 70 to 90 percent of maximum (Tolman et al., 1978).

The equipment types used to construct graded slopes consist of both standard and specialized landfill vehicles. Excavation, hauling, spreading, and compaction of cover materials are the major elements of a complete grading operation. Landfill grading vehicles include crawler dozers and loaders, rubber-tired dozers and loaders, landfill compactors, and scrapers (Figure 3-3). Table 3-3 summarizes the cover material handling capabilities of these equipment types.

Crawler tractors (dozers and loaders) are excellent landfill grading machines (Brunner and Keller, 1972). Dozers are fitted with U-shaped landfill

TABLE 3-3

COVER MATERIAL HANDLING CHARACTERISTICS OF LANDFILL EQUIPMENT[1]

Equipment	Rating[2] for Excavating	Spreading	Compacting	Hauling
Crawler dozer	E	E	G	NA
Crawler loader	E	G	G	NA
Rubber-tired dozer	F	G	G	NA
Rubbertired loader	F	G	G	NA
Landfill compactor	P	G	E	NA
Scraper	G	E	NA	E

[1] Assuming easily workable soil and cover material haul distance 1,000 ft.
[2] Rating key: E, excellent; G, good; F, fair; P, poor, NA, not applicable.
(Source: Brunner and Keller, 1972)

blades for pushing and spreading; loaders can be fitted with landfill blades and multi-purpose buckets for excavation and spreading.

Rubber-tired dozers and loaders are generally faster and more agile than crawler machines; however, they do not excavate as well. Rubber-tired dozers are used infrequently at landfill sites, because they do not grade as well as crawler dozers on the irregular and spongy surface provided by compacted refuse (Brunner and Keller, 1972).

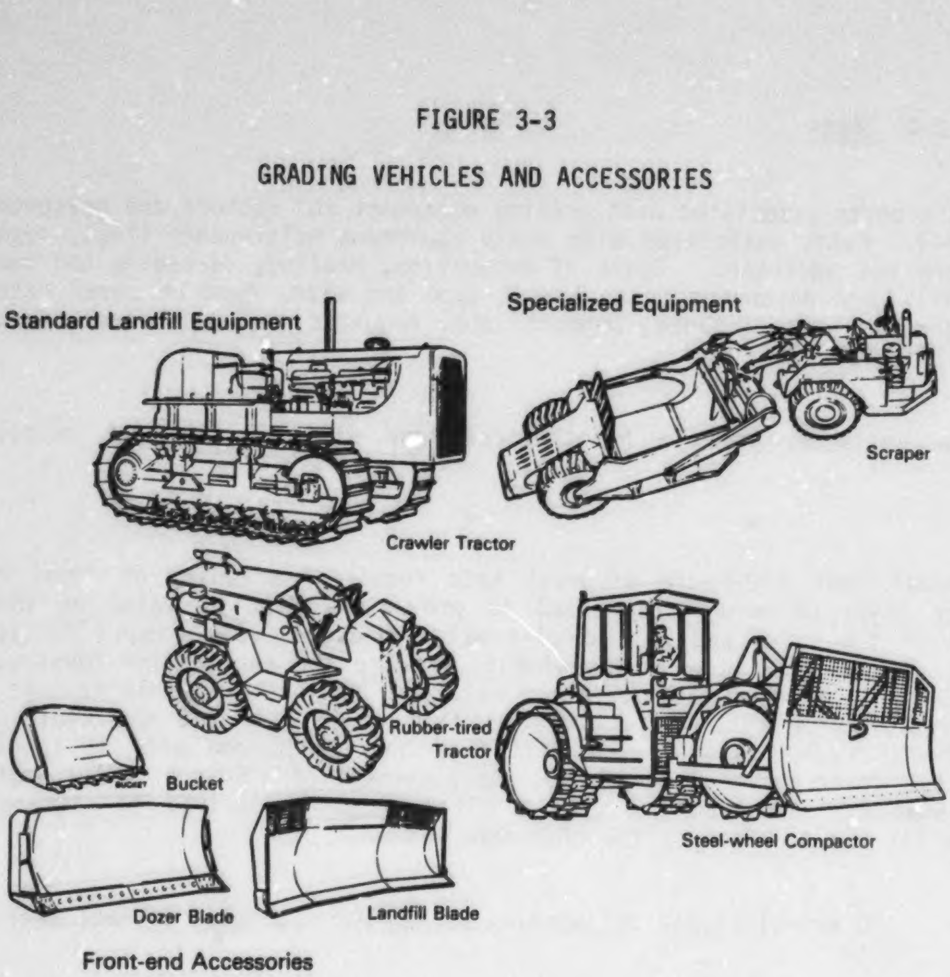
Specialized landfill vehicles include compactors and scrapers. Steel-wheeled landfill compactors are excellent machines for spreading and compacting on flat to moderate slopes. Scrapers are effective in excavating, hauling, and spreading cover materials over relatively long distances (Brunner and Keller, 1972).

3.2.3 Advantages and Disadvantages

Surface grading of covered disposal sites, when properly designed and constructed to suit individual sites, can be an economical method of controlling infiltration, diverting runoff, and minimizing erosion. A properly sealed and graded surface will aid in the reduction of subsurface leachate formation by minimizing infiltration and promoting erosion-free drainage of

FIGURE 3-3

GRADING VEHICLES AND ACCESSORIES


Standard Landfill Equipment
Specialized Equipment
Scraper
Crawler Tractor
Rubber-tired Tractor
Bucket
Steel-wheel Compactor
Dozer Blade
Landfill Blade
Front-end Accessories

surface runoff. Grading can also be used to prepare a cover soil capable of supporting beneficial plant species.

There may, however, be certain disadvantages associated with grading the surface of a given site. Large quantities of a difficult-to-obtain cover soil may be required to modify existing slopes. Suitable sources of cover material may be located at great distances from the disposal site, increasing hauling costs. Also, periodic regrading and future site maintenance may be necessary to eliminate depressions formed through differential settlement and compaction, or to repair slopes that have slumped or become badly eroded (Tolman et al., 1978).

3.2.4 Costs

Unit costs associated with grading equipment and methods are presented in Table 3-4. Costs associated with heavy equipment maintenance (fuel, repairs, etc.) are not addressed. Costs of excavation, hauling, spreading and compaction will vary depending on equipment type and size, type of cover material being graded, haul distance, support labor required, and unforeseen construction difficulties.

The costs to grade a hypothetical land disposal site are calculated below.

* * *

Assume that a 20-acre disposal site requires regrading of final cover (12-inch layer of sandy clay loam) to prevent ponding of water on the top portion of the site, and to control erosion on excessively steep side slopes. Grading at the site is also intended to prepare the surface for construction of a 12-inch clay cap. Assume that final cover soil is available on-site, but that regrading of 15 surface acres of this cover fill and the addition of 200,000 ft^3 (7,400 yd^3) of new fill from on-site borrow pits is required. Once final cover has been re-graded, the placement of a 6-inch buffer layer of sand (available from borrow site 2 miles away) is desired to prepare the surface for construction of the clay cap.

- (20 acres) (4,840 yd^2/acre) = 96,800 yd^2 ... total surface area of site
- (15 acres) (4,840 yd^2/acre) = 72,600 yd^2 ... surface area of cover soil to be re-graded
- 12-inch final cover = (1/3 yd) 72,600 yd^2) = 24,200 yd^3 ... volume of cover soil to be re-graded
- 24,200 yd^3 + 7,400 yd^3 (additional fill required) = 31,600 yd^3 ... total volume of final cover to be graded
- (31,600 yd^3) (\$1.50/yd^3) = \$47,400 ... cost to re-grade final cover at site (from range reported in McMahon and Pereira, 1979)
- 6-inch layer of sand = (0.167 yd) (96,800 yd^2) = 16,166 yd^3 ... volume of sand required from 2 miles off-site
- (16,166 yd^3) (2.00/yd^3) = \$32,330 ... cost of hauling, spreading, compacting sand
- Total cost to regrade 20-acre site and to prepare site for clay cap:
  = \$47,400 + \$32,330
  = \$79,730 or approximately \$80 thousand

Regrading is a relatively inexpensive remedial action when suitable cover materials are available on-site or close to the disposal site.

TABLE 3-4

UNIT COSTS ASSOCIATED WITH GRADING COVERED DISPOSAL SITES

Description	Unit cost	Source
● Topsoil (sandy loam), hauling, spreading, and grading (within 20 miles); labor, materials, and equipment	$\$13/\text{yd}^3$	1
● On-site excavation, hauling, spreading, and compaction of earth (1,000) - 5,000' haul); labor and equipment	$\$.94 - 2.06/\text{yd}^3$	2
● Loam topsoil; material only	$\$5/\text{yd}^3$	2
● Excavate, haul 2 miles, spread and compact loam, sand, or loose gravel (with front and end loader); labor and equipment only	$\$1.75 - 2.00/\text{yd}^3$	2
● Grading site excavation and fill (no compaction)		3
75 h.p. dozer, 300' haul	$\$2.29/\text{yd}^3$	
300 h.p. dozer, 300' haul	$\$1.54/\text{yd}^3$	
● Testing soils for compaction	\$27.50/sample	2

---

[1] Haseley Trucking Company, 1980.
[2] McMahon and Pereira, 1979.
[3] Godfrey, 1979.

3.3 REVEGETATION

The establishment of a vegetative cover may be a cost-effective method to stabilize the surface of hazardous waste disposal sites, especially when preceded by surface sealing and grading. Revegetation decreases erosion by wind and water and contributes to the development of a naturally fertile and stable surface environment. Also, the technique can be used to upgrade the appearance of disposal sites that are being considered for various re-use options. Short-term vegetative stabilization (i.e., on a semi-annual or seasonal basis) can also be used as a remedial technique for polluting disposal sites.

3.3.1 Description and Applications

Revegetation may be part of a long-term site reclamation project, or it may be used on a temporary or seasonal basis to stabilize intermediate cover surfaces at waste disposal sites. Revegetation may not be feasible at disposal sites with high cover soil concentrations of phytotoxic chemicals, unless these sites are properly sealed and vented and then recovered with suitable topsoil. A systematic revegetation plan will include: (1) selection of suitable plant species, (2) seedbed preparation, (3) seeding/planting, (4) mulching and/or chemical stabilization, and (5) fertilization and maintenance.

Long-term vegetative stabilization generally involves the planting of grasses, legumes, and shrubs. The establishment of short-term, seasonal vegetative cover is limited principally to species of grasses. The selection of suitable plant species for a given disposal site depends on several site-specific variables. Species selection criteria are discussed in Section 3.3.2.

Grasses such as fescue and lovegrass provide a quick and lasting ground cover, with dense root systems that anchor soil and enhance infiltration. Legumes (lespedeza, vetch, clover, etc.) store nitrogen in their roots, enhancing soil fertility and assisting the growth of grasses. They are also readily established on steep slopes. Shrubs such as bristly locust and autumn olive also provide a dense surface cover, and certain species are quite tolerant of acidic soils and other possible disposal site stresses. Trees are generally planted in the later stages of site reclamation, after grasses and legumes have established a stable ground cover. They help provide long-term protective cover and build up a stable, fertile layer of decaying leaves and branches. A well-mixed cover of grasses, shrubs, and trees will ultimately restore both economic and aesthetic value to a reclaimed site, providing suitable habitat for populations of both humans and wildlife. Tables 3-5 and 3-6 summarize the suitability of various plant species for revegetation purposes.

Seedbed preparation is necessary to ensure rapid germination and growth of the planted species. Applications of lime will help neutralize highly acidic topsoils. Similarly, fertilizers should be added for cover soils low in essential plant nutrients. Optimum soil application rates for lime and fertilizers should be determined from site-specific soil tests. Where re-

TABLE 3-5

CHARACTERISTICS OF GRASSES AND LEGUMES COMMONLY USED FOR REVEGETATION

Common name	Season			Site suitability			Growth habit[1]	Optimum pH range
	Cool	Warm	Dry	Well drained	Moderately drained	Poorly drained		
GRASSES								
Bahiagrass		x	x	x	x		P	4.5 - 7.5
Barley	x			x	x		A	5.5 - 7.8
Bermuda grass		x	x	x	x		P	5.5 - 7.5
Bluegrass, Canada	x		x	x	x		P	4.5 - 7.5
Bluegrass, Kentucky	x			x	x		P	5.5 - 7.0
Bluestem, Big		x		x	x		P	5.0 - 7.5
Bromegrass, Field	x			x	x		A	6.0 - 7.0
Bromegrass, Smooth	x		x	x	x		P	5.5 - 8.0
Buffalo grass		x			x		P	6.5 - 8.0
Canary grass, Reed	x		x	x	x	x	P	5.0 - 7.5
Deer tongue		x	x	x	x	x	P	3.8 - 5.0
Fescue, tail	x			x	x		P	5.0 - 8.0
Grama, blue		x	x	x	x		P	6.0 - 8.5
Indian grass		x			x		P	5.5 - 7.5
Lovegrass, sand		x		x			P	6.0 - 7.5
Lovegrass, weeping		x	x	x	x		P	4.5 - 8.0
Oats	x		x	x			A	5.5 - 7.0
Orchard grass	x		x	x	x		P	5.0 - 7.5
Red top	x		x	x	x	x	P	4.0 - 7.5
Ryegrass	x		x	x			A,P	5.5 - 7.5

--continued--

TABLE 3-5 (Continued)

Common name	Season			Site suitability			Growth habit[1]	Optimum pH range
	Cool	Warm	Dry	Well drained	Moderately drained	Poorly drained		
GRASSES (con't)								
Switchgrass		x	x	x			P	5.0 - 7.5
Timothy	x			x	x	x	P	4.5 - 8.0
Wheatgrass, Tall	x		x	x	x	x	P	6.0 - 8.0
Wheatgrass, Western	x		x	x	x	x	P	4.5 - 7.0
LEGUMES								
Alfalfa	x		x	x			P	6.5 - 7.5
Clover, Alsike	x		x	x	x	x	P	6.0 - 7.0
Lespedeza, Common			x	x	x		A	5.0 - 6.0
Lespedeza, Sericea		x	x	x	x		P	5.0 - 7.0
Sweet Clover, White or Yellow	x		x	x	x		B	6.0 - 8.0
Trefoil, Birdsfoot	x		x	x	x		P	5.0 - 7.0
Vetch, Crown	x		x	x	x		P	5.5 - 7.5
Vetch, Harry	x		x	x	x		A	5.0 - 7.5

---

[1] A = annual, B = biennial, P = perennial

(Source: EPA, 1976)

TABLE 3-6

COMMONLY USED TREES AND SHRUBS

Common name	Remarks
Shrubs	
● Amur honeysuckle	Good for wildlife. Increased vigor and adaptability as plants mature.
● Bristly locust	Extreme vigor. Thicket former. Good erosion control. Rhizomatous, 5-7 ft. tall. Excellent on flat areas and outslopes.
● Autumn-olive	Nitrogen-fixing. Good for wildlife. Excellent fruit crops. Wide adaptation. Up to 15 ft. tall.
● Indigo bush	High survival on acid soil. Leguminous. Thicket former. Slow spreader. 8-12 ft. tall.
● Japanese fleece flower	Good growth on many sites, especially moist areas. Excellent leaf litter and canopy cover, pH range, 3.5-7.0.
● Tatarian honeysuckle	Upright shrub, forms clumps. Grows well on well-drained soils. Up to 10 ft. tall. Takes 2 years for good cover.
Trees, conifers	
● Virginia pine	Tolerant of acid soil. Slow development. Good for wildlife.
● Pitch pine	Deep rooted, very acid tolerant. Can survive fire injury. Attractive to deer. Plant in bands or blocks.
● Loblolly pine	Very promising. Rapid early growth. Marketable timber. Can survive pH 4.0 to 7.5. Susceptible to ice/snow damage.
● Japanese larch	Plant on unleveled, noncompacted soil. Provides good litter.

--continued--

TABLE 3-6 (Continued)

Common name	Remarks
● Eastern red cedar	Tall, narrow growth. Best on dry, sandy soils. Good with black locust. pH 5.0. to 8.0.
● Mugho pine	Survives on acid spoil. Develops slowly. Low growing. Good wildlife cover.
Trees, hardwoods	
● Black locust	Can be direct-seeded. Wide adaptation. Rapid growth; good leaf litter. Use mixed planting.
● Cottonwood	Desirable for larger-scale planting. Good cover; rapid growth. Plant pure stands.
● European black alder	Rapid growth; wide adaptation. Nitrogen fixing. Survives pH 3.5 to 7.5. Adapted to all slopes.
● Green ash	Very promising. Use on all slopes and graded banks with compact loams and clays. Plant in hardwood mixture.
● Hybrid poplar	Rapid growth. Good survival at low pH. Marketable timber after 20 years. Cannot compete with grasses.
● Red oak	Slow initial growth. Good survival; plant on upper and lower slopes only. pH 4.0 to 7.5.
● Sycamore	Very desirable for planting. Volunteer trees grow faster than planted ones.

(Source: EPA, 1976)

quired, lime should be determined from site-specific soil tests. Where required, lime should be worked to 6-inch depths into the soil by discing or harrowing (EPA, 1976). For dense, impervious topsoils, loosening by tillage is recommended. Grading methods for loosening and roughening large areas of soils are discussed in Section 3.2.1.

Seeding should be performed as soon as possible after final grading and seedbed preparation (EPA, 1976). The most common and efficient method of seeding large areas of graded slopes is with hydroseeders. Seed, fertilizer, mulch, and lime can be sprayed from hydroseeders onto steep outslopes and other areas of difficult access. Rear-mounted blowers can be attached to lime trucks to spread seed and fertilizers over such areas, also. Grass or grain drills may be used to apply seed on gently rolling or level, stone-free terrain. Hand planting, a time-consuming and costly project, may be required for trees and shrubs (EPA, 1976).

Mulches or chemical stabilizers may be applied to seeded soils to aid in the establishment of vegetative cover. Organic mulches such as straw, hay, wood chips, saw dust, dry bark, bagasse (unprocessed sugar cane fibers), excelsior (fine wood shavings), and manure protect bare seedbed slopes from erosion prior to germination. Also, thin blankets of burlap, fiberglass, and excelsior can be stapled down or applied with asphalt tacks to form protective mulch mats for germinating seedbeds.

Mulches conserve soil moisture, dissipate raindrop energy, moderate soil temperatures, prevent crusting, increase infiltration, and generally control wind and water erosion. Mulches are usually applied after seeding and fertilization, although certain mulch materials (e.g., wood fibers) may be applied as hydroseeder slurries mixed with seed, fertilizer, and lime (EPA, 1976). Mulch application rates will vary depending on local climate, soil characteristics, and slope steepness.

Loose straw and hay mulches are the most common and most cost-effective temporary soil stabilizer/mulching materials available (EPA, 1976). These mulches are best applied using a mulch blower, at rates from 1/2 to 4 tons per acre. Straw/hay mulches can be anchored to the soil by asphalt, chemical binders, or jute netting (EPA, 1976).

Chemical stabilizers are binders and tacks that are sprayed on bare soils or mulches to coat, penetrate, and bind together the particles. Stabilizers reduce soil water loss and enhance plant growth by temporarily stabilizing seeded soils against wind and water erosion. They can also be used to stabilize graded soils in the off season until spring seeding. Stabilizers are used extensively in arid regions to help dry, permeable soils retain soil moisture (EPA, 1976).

Chemical soil stabilizers include latex emulsions, plastic films, oil-in-water emulsions, and resin-in-water emulsions. Table 3-7 summarizes pertinent characteristics of seven commercially available stabilizers, including cost data (where available).

In field tests comparing the effectiveness of these chemical additives in controlling erodibility of several regional soil types in Virginia, none of the stabilizers tested were determined to be as cost-effective as conventional mulches of straw and asphalt-emulsions (Lutton et al., 1979).

Periodic reliming and fertilization may be necessary to maintain optimum yearly growth on seeded plots. Soils with poor buffering capacity may require frequent liming to achieve suitable pH levels; these are generally soils high in organic matter or clay content. Annual fertilization of nitrogen-, phosphorus-, or potassium-deficient soils will also aid reclamation efforts. Fertilizer application rates will vary with the nutrient content and pH level of the seeded cover soil. Twice yearly mowing and the judicious use of selective herbicides will help control undesirable weed and brush species. Grass sodding and remulching or planting new shrubs and trees is recommended for sparsely covered, erosion-prone areas (Lutton et al., 1979).

3.3.2 Design and Construction Considerations

The selection of suitable plant species for purposes of revegetating a given disposal site will depend on cover soil characteristics (grain size, organic content, nutrient and pH levels, and water content), local climate, and site hydrology (slope steepness and drainage characteristics). Individual species must be chosen on the basis of their tolerance to such site-specific stresses as soil acidity and erodibility and elevated levels of landfill gases or phytotoxic waste components (e.g., heavy metals, salts) in cover soil. Other important considerations include the species compatibility with other plants selected to be grown on the site, resistance to insect damage and diseases, and suitability for future land use (EPA, 1976).

Long-term vegetative stabilization and site reclamation require the proper planting of compatible mixes of grasses, legumes, shrubs, and trees. Short-term revegetation efforts--for instance, for remedial action at active disposal sites that have yet to be sealed--generally require the use of low-cost, quick-growing perennial and self-seeding annual species, usually grasses (EPA, 1979).

The optimum time for seeding depends on local climatic considerations and the individual species adaptations. For most perennial species in most localities, early fall seeding is recommended. Annuals are usually best seeded in spring and early summer, although they can be planted for quick vegetation whenever soil is damp and warm (Lutton et al., 1979). In mild climates (e.g.,

TABLE 3-7

SUMMARY OF CHEMICAL STABILIZERS FOR COVER SOIL

	Uses			Erosion Resistance			
Name	Soil Stabilizer	Mulch	Mulch Tack	Water	Dust/Wind	Description	Product Information
Aerospray® 52	x	x		x		Water dispersible, alkyd resin emulsion; forms hard crust; nontoxic nonphytotoxic; pH 8-9; ∿$2.85/gal	American Cyanamid Co., Industrial Chemicals and Plastic Div. Wayne, NJ 07970
Aerospray® 70	x	x	x	x		Water dispersible, polyvinyl acetate resin emulsion; effective in sand; ∿$2.50/gal	American Cyanamid Co., Industrial Chemicals and Plastic Div. Wayne, NJ 07970
Aquatain	x	x		x		Water dispersible, concentrate of chemicals and pectin; forms fragile crust; nontoxic; nonflammable; ∿$2.30/gal	Larutan Corp., Anaheim, CA 02805
Curasol® AE	x	x	x		x	Water dispersible, polyvinyl acetate latex emulsion;	American Hoechst Corp., Bridgewater, NJ 08876

--continued--

TABLE 3-7 (Continued)

Name	Uses			Erosion Resistance		Description	Product Information
	Soil Stabilizer	Mulch	Mulch Tack	Water	Dust/Wind		
Curasol® AE (continued)						hard crust; nontoxic; nonphytotoxic; pH 4-5; ~$2.60/gal	
Curasol® AH	x		x		x	Water dispersible, high polymer synthetic resin; flexible crust, nontoxic; nonphytotoxic; pH 4-5	American Hoechst Corp., Bridgewater, NJ 08876
DCA - 70	x	x	x		x	Water dispersible, polyvinyl acetate emulsion; can be reinforced with fiberglass filaments; nontoxic; nonphytotoxic; nonflammable; pH 4-6	Union Carbide Corp., Chemicals and Plastics New York, NY 10017
Petroset® SB	x	x	x	x	x	Water dispersible oil emulsion; effective in particles below gravel size; nontoxic; nonflammable; pH 6 ±0.5; ~$1.60/gal	Phillips Petroleum Co. Chemical Dept., Bartlesville, OK 74003

(Sources: Lutton et al., 1979 [costs], and EPA, 1976.)

southeastern United States) the growth of both summer and winter grasses will extend the range of evapotranspiration and erosion resistance for cover soils (Lutton et al., 1979).

A recent study of vegetative growth in landfill environs (Gilman et al., 1979) ranked the relative tolerance of selected tree species to high cover soil concentrations of CO_2 and methane, a typical landfill condition that causes the displacement of oxygen from the root zones of growing plants. The tree species included (in order of decreasing tolerance): Black Gum, Norway Spruce, Gingko, Black Pine, Bayberry, Mixed Poplar, White Pine, Pin Oak, Japanese Yew, American Basswood, American Sycamore, Red Maple, Sweet Gum, Green Ash, and Honey Locust. The same study found that mounds of topsoil underlain by clay gas-barriers, or trenches underlain with polyethylene sheeting and vented with perforated PVC vent pipes (Figures 3-4, A and B) effectively prevented the migration of landfill gases (products of anaerobic decomposition) into the root zone of trees. A further discussion on gas migration control can be found in Section 6.0. The Gilman study also concluded that woody plant species are more likely to survive on a completed fill if planted when small, generally less than three feet tall (Gilman, et al., 1979).

Loamy topsoils--those with nearly equal percentages of clay, silt, and sand-sized grains--are generally best suited for revegetation establishment. They are easily seeded and allow easy root penetration.

Sandy soils may be productive when blended or mulched with organic matter (Lutton et al., 1979).

3.3.3 Advantages and Disadvantages

A well-designed and properly implemented revegetation plan--whether for long-term reclamation or short-term remedial action--will effectively stabilize the surface of a covered disposal site, reducing erosion by wind and water, and will prepare the site for possible reuse. Evapotranspiration and interception of precipitation by vegetative cover will also control leachate generation at landfills by drying out the water near surface layers of refuse and soil (Molz et al., 1974). This effect, however, is more or less offset by enhanced soil infiltration capacity due to the increased detention of surface flow by the vegetation and to the effects of the root systems on the cover soil (increases permeability). If subsurface liners of clay or synthetic membranes are constructed, infiltration of water into buried wastes (and subsequent leachate production) will virtually be eliminated. This illustrates the importance of a layered surface sealing system and properly graded slopes, which, in combination with suitable vegetative cover, will isolate buried wastes from surface hydrologic input.

FIGURE 3-4A

CROSS SECTION END VIEW OF GAS BARRIER TRENCH

(Source: Gilman et al., 1979)

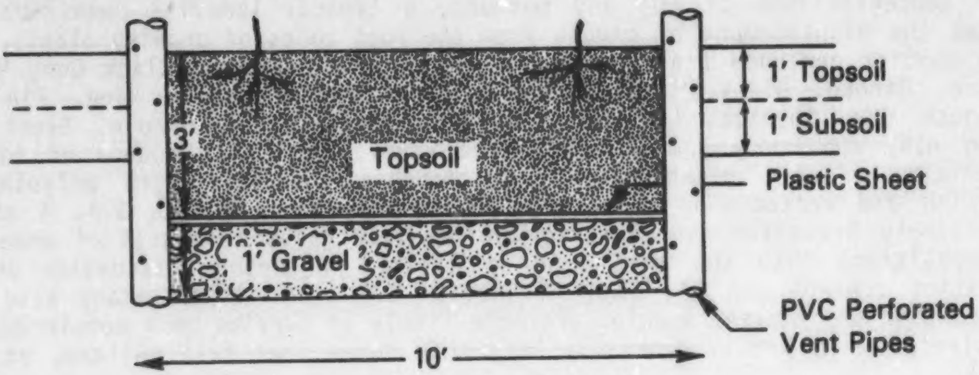
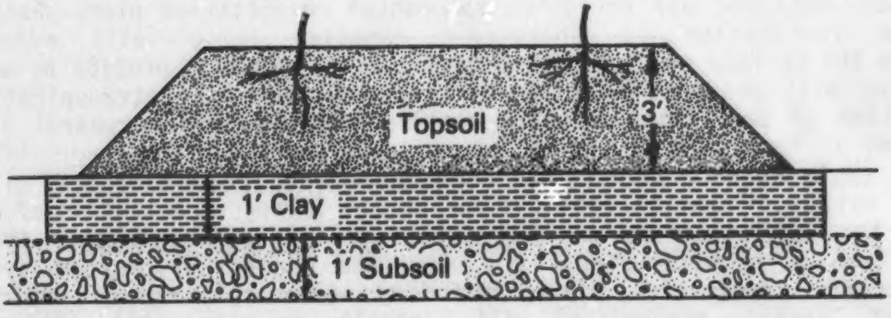

1' Topsoil
1' Subsoil
3'
Topsoil
Plastic Sheet
1' Gravel
PVC Perforated Vent Pipes
10'

FIGURE 3-4B

CROSS SECTION END VIEW OF SOIL MOUND

(Source: Gilman et al., 1979)


Topsoil
3'
1' Clay
1' Subsoil

The selection of suitable plant species, the use of appropriate mulches and stabilizers, the application of required doses of lime and fertilizers, and optimum timing in seeding will help ensure the establishment of an effective vegetative cover. However, unforeseen difficulties may compromise the effectiveness of revegetation. Clays or synthetic barriers below supporting topsoil in poorly drained areas may cause swamping of cover soil and subsequent anaerobic conditions. Too thin a cover soil may dry excessively in arid seasons and irrigation may be necessary. Improperly vented gases and soluble phytotoxic waste components may kill or damage vegetation. The roots of shrubs or trees may penetrate the waste cover and cause leaks of water infiltration and gas exfiltration. Also, periodic maintenance of revegetated areas--liming, fertilizing, mowing, replanting, or regrading eroded slopes--will add to the costs associated with this remedial technique.

3.3.4 Costs

Table 3-8 presents various unit costs associated with revegetating covered disposal sites. These data represent estimated 1980 costs for the indicated revegetation activities.

The costs of revegetating a hypothetical 20-acre disposal site are calculated from these unit costs.

Assume that a 20-acre disposal site has been capped and graded, and has been prepared for revegetation (loam topsoil has been spread and tilled). The entire site is to be hydroseeded in the spring with grass seed, lime, and fertilizer. It is to be hay-mulched over 15 acres; 5 acres of the site are relatively steep slope areas that require stapled jute mesh for erosion control. Assume that the site will require six grass mowings the first year and one refertilizing operation. Assume that no other maintenance (e.g., sodding) is required.

The site is to be planted with selected evergreens and shrub species in the spring of the second year of revegetation. Assume that 1,000 30- to 36-inch-high evergreens and 1,000 2- to 3- feet-high shrubs are to be evenly distributed over the site. Each plant requires handspread wood chip mulching over 1 ft^2 (.111 yd^2) at its base.

TABLE 3-8

UNIT COSTS ASSOCIATED WITH REVEGETATION OF COVERED DISPOSAL SITES

Description	Unit costs[1]	Source
Hydraulic spreading (hydroseeding), lime, fertilizer, and seed	$\$470/\text{acre}$	2
Mulching, hay	$\$143/\text{acre}$	2
Loam topsoil, remove and stockpile on-site; using 200 h.p. dozer, 6' deep, 200' haul	$\$ .76/\text{yd}^3$	3
500' haul	$\$2.92/\text{yd}^3$	3
Hauling loam on-site	$\$1.86/\text{yd}^3$	2
Spreading loam, 4-6" deep	$\$ .40 - .70/\text{yd}^3$	2
Plant bed preparation (unspecified), 18" deep, by machine	$\$6.21/\text{yd}^2$	3
Hydraulic seeding and fertilization of large areas, with wood fiber mulch	$\$ .33/\text{yd}^2$	3
Mulch, hand spread 2" deep, wood chips	$\$1.01/\text{yd}^2$	3
Liming slope areas	$\$220/\text{acre}$	2
Fertilizing, level	$\$200/\text{acre}$	2
slope	$\$270/\text{acre}$	2
Seeding, level	$\$325/\text{acre}$	2
slope	$\$395/\text{acre}$	2
Jute mesh, stapled (erosion control)	$\$ .65/\text{yd}^2$	3
Sodding, in East, 1" deep, level	$\$3.10/\text{yd}^2$	3
slope	$\$3.42/\text{yd}^2$	
Maintenance:		
Grass mowing, slopes	$\$38/\text{acre}$	2
level areas	$\$17/\text{acre}$	2
Refertilization	$\$128/\text{acre}$	2
Weeding/pruning shrubs	$\$810/\text{acre}$	2

--continued--

TABLE 3-8 (Continued)

Description		Unit costs[1]	Source
On-site planting			
● Trees, evergreens	30-36"	$44 ea.	2
	36-42"	$56 ea.	2
	42-48"	$80 ea.	2
	4-5'	$100 ea.	2
	5-6'	$130 ea.	2
Black Pines	7-8'	$130 ea.	3
Yews	2-2.5'	$33 ea.	3
Junipers	4-5'	$44 ea.	3
● Shade trees (balled and burlapped)			
	6-8'	$37 ea.	2
	8-10'	$52 ea.	2
	1.5 - 2.5" diam.	$140-$220 ea.	2
	2.5 - 4.0" diam.	$350-$500 ea.	2
Birch	8-10'	$78 ea.	3
Oak	8-10'	$84 ea.	3
● Shrubs (balled and burlapped)			
	2-3'	$18 ea.	2
	3-4'	$41 ea.	2
	4-5'	$50 ea.	2
Honeysuckle shrub	4-5'	$29 ea.	3

---

[1] All costs include materials and installation (labor and equipment), unless otherwise indicated. Note different units (acre; yd^2; yd^3; each).

[2] McMahon and Pereira, 1979.

[3] Godfrey, 1979.

The site is to be further planted in its third year of reclamation with selected species of 6- to 8- feet-high shade trees. Assume that 500 of these trees are to be planted, and that no special maintenance (mulching, pruning) will be required.

Site Revegetation Costs:

First Year

o Hydroseeding with seed, lime, and fertilizer: ($470/acre) (20 acres) = $9,400

- Hay mulching over 15 acres: (\$143/acre) (15 acres) = \$2,145

- Jute mesh over 5 acres: (5 acres) (4,840 yd^2/acre) (\$0.65/yd^2) = \$15,730

- Grass mowings, 6 times: (20 acres) (\$38/acre) (6 mowings/year) = \$4,560/yr.

- Refertilizing, 1 time: (20 acres) (\$138/acre) = \$2,760

- TOTAL FIRST-YEAR COSTS = \$34,600

Second Year

- Evergreens: (1,000 plants) (\$44/planting) = \$44,000

- Shrubs (balled and burlapped): (1,000 plants) (\$18/planting) = \$18,000

- Hand-spread wood chip mulching: (2,000 plants) (.111 yd^2/plant) (\$1.01/yd^2) = \$224

- TOTAL SECOND-YEAR COSTS = \$62,200

Third Year

- Shade trees: (500 plants) (\$37/planting) = \$18,500

TOTAL REVEGETATION COSTS OVER 3 YEARS (including one year of maintenance) = \$115,300

3.4 SURFACE WATER DIVERSION AND COLLECTION

The construction of surface water diversion and collection structures may provide short-term or permanent measures to hydrologically isolate waste disposal sites from surface inputs. Surface runoff can be managed so that it does not contribute to leachate generation or erosion of cover materials. Conventional civil engineering techniques are used to control flooding, surface water infiltration, and off-site erosive transport of possible contaminated sediments and debris.

Several well-established construction techniques are available for diverting and handling surface water flow in critical areas. Those methods most applicable as remedial measures at uncontrolled disposal sites are addressed below.

3.4.1 Dikes and Berms

3.4.1.1 Description and Applications

Dikes and berms are well-compacted earthen ridges or ledges constructed immediately upslope from or along the perimeter of disturbed areas (e.g., disposal sites). These structures are generally designed to provide short-term protection of critical areas by intercepting storm run-off and diverting the flow to natural or manmade drainage ways, to stabilized outlets, or to sediment traps. The two terms, dikes and berms, are generally used interchangeably; however, dikes may also have applications as flood containment levees (Section 3.4.6).

Dikes and berms may be used to prevent excessive erosion of newly constructed slopes until more permanent drainage structures are installed or until the slope is stabilized with vegetation (EPA, 1976). Dikes and berms will help provide temporary isolation of un-capped and unvegetated disposal sites from surface run-off that may erode the cover and infiltrate the fill. These temporary structures are designed to handle relatively small amounts of runoff; they are not recommended for unsloped drainage areas larger than 5 acres (Virginia SWCC, 1974).

3.4.1.2 Design and Construction Considerations

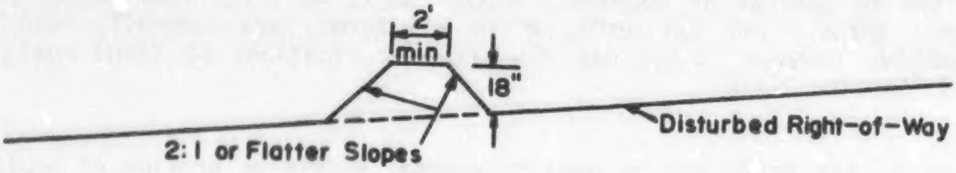
Specific design and construction criteria for berms and dikes will depend upon desired site-specific functions of the structures. An interceptor dike/berm (Figure 3-5) may be used solely to shorten the length of exposed slopes on or above a disposal site, thereby reducing erosion potential by intercepting and diverting runoff. Diversion dikes/berms (Figure 3-6) may be installed at the top of the steeper side slopes of unvegetated disposal sites to provide erosion protection by diverting runoff to stabilized channels or outlets.

Dikes and berms ideally are constructed of erosion-resistant, low-permeability, clayey soils. Compacted sands and gravel, however, may be suitable for interceptor dikes and berms. The general design life of these structures is on the order of one year maximum; seeding and mulching or chemical stabilization of dikes and berms may extend their life expectancy. Stone stabilization with gravel or stone rip-rap immediately upslope of diversion dikes will also extend performance life.

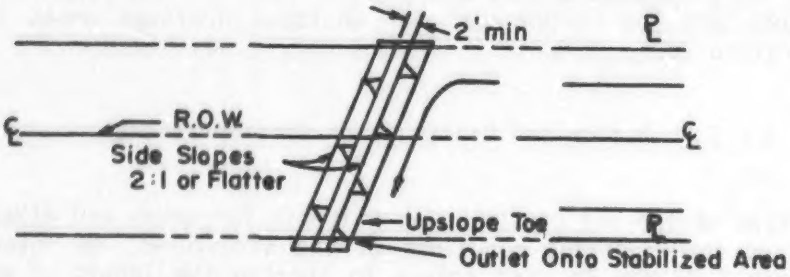
FIGURE 3-5

TEMPORARY INTERCEPTOR DIKE

(Source: Virginia SWCC, 1974)


2'
min
18"
2:1 or Flatter Slopes
Disturbed Right-of-Way

CROSS SECTION

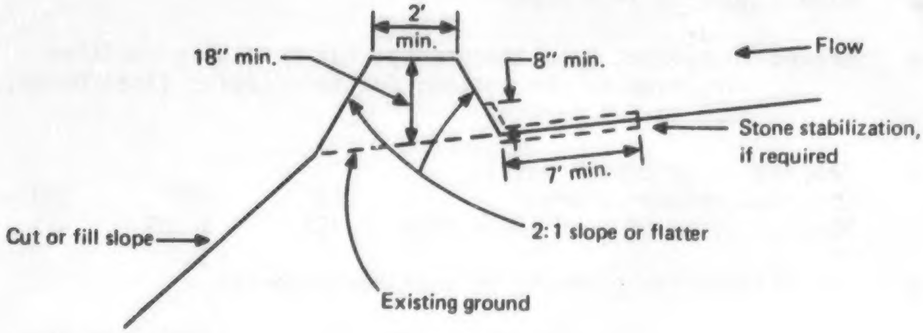

2' min
℄
R.O.W.
℄
℄
Side Slopes
2:1 or Flatter
Upslope Toe
℄
Outlet Onto Stabilized Area

PLAN VIEW

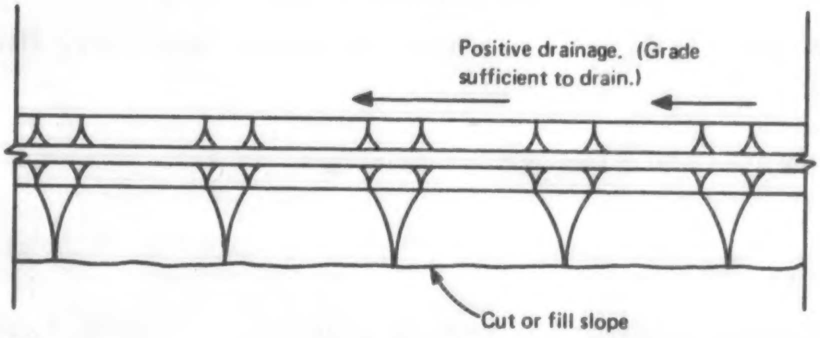
FIGURE 3-6

TEMPORARY DIVERSION DIKE

(Source: U.S. EPA, 1976)


2' min.
18" min.
8" min.
Flow
Stone stabilization, if required
7' min.
Cut or fill slope
2:1 slope or flatter
Existing ground

Cross-section


Positive drainage. (Grade sufficient to drain.)
Cut or fill slope

Plan view

Common design and construction criteria for interceptor and diversion dikes/berms include the following:

- Drainage area: 5 acres maximum
- Top width: 2 feet minimum
- Height (compacted): 18 inch minimum
- Side slopes: 2:1 or flatter
- Grade: Dependent upon topography, but must have positive drainage to the outlet; for interceptor dikes/berms, 1.0 to 1.5 percent.
- Spacing (interceptor only):

Distance between dikes	150'	200'	300'
Maximum slope of area above dike	>10%	5-10%	<5%

- All earthen dikes should be machine-compacted.
- Diverted runoff should outlet directly onto stabilized areas, level spreader, grassed channel, or chute/downpipe.
- Periodic inspection and maintenance should be provided.
- Diversion dikes must be seeded and mulched immediately after construction.

3.4.1.3 Advantages and Disadvantages

The following advantages are afforded by properly designed and constructed dikes and berms:

- Standard construction techniques are used; required excavation and grading equipment may be available at disposal site.
- Required earth fill may be available on-site.
- Erosion of cover material from slopes of disposal site will be controlled until further site stabilization is achieved (through cover compaction, regrading, revegetation).
- Diversion of storm runoff will decrease amount of water available for infiltrating soil cover; therefore, leachate generation may be reduced.

Disadvantages associated with berms and dikes are generally due to maintenance requirements and the nonpermanent nature of the structures. These problems include the following:

- Periodic inspections and maintenance are required to ensure structural integrity and prevent upslope deposition of sediments.
- Improperly installed dikes and berms may increase seepage, contributing to soil instability and leachate generation.
- The structures are suitable only for relatively small drainage areas (5 acres).
- Ultimate removal of the structures (after effective performance life) entails additional costs.

3.4.1.4 Costs

The costs of installing and maintaining dikes and berms will vary with the site conditions, depending on: number and size of structures required, local availability of suitable soil and equipment, local climate and site hydrology (intensity and volume of storm runoff to be diverted), amount of maintenance required, design life of the structures, amount and type of stabilization required (seeding, mulching, chemical soil additives, and unforeseen construction difficulties.

Unit costs associated with dike/berm construction and maintenance are presented in Table 3-15 at the end of this chapter.

3.4.2 Ditches, Diversions, and Waterways

3.4.2.1 Description and Applications

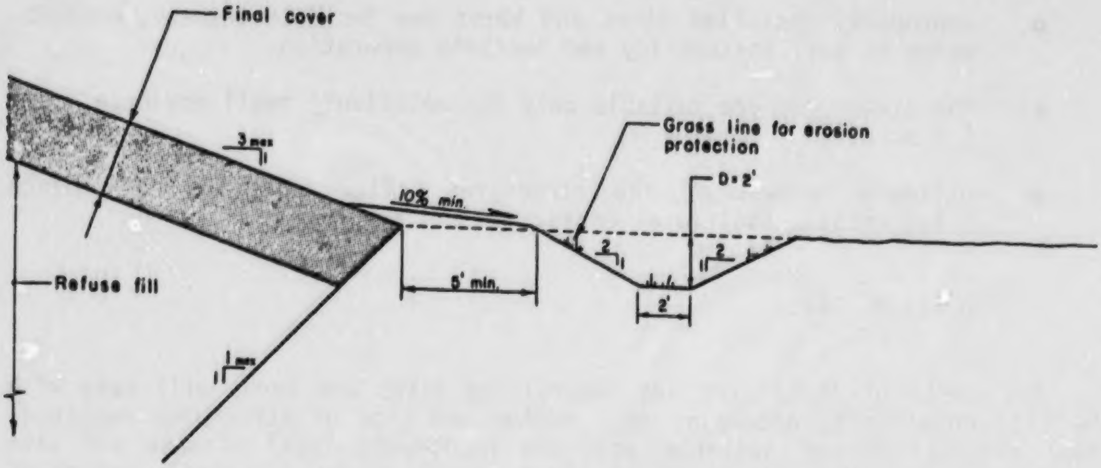
Ditches (or swales) are excavated, temporary drainageways used above and below disturbed areas to intercept and divert runoff. They may be constructed along the upslope perimeter of disposal areas to intercept storm runoff and carry it to natural drainage channels downslope of the site. As shown in Figure 3-7, ditches may also be installed downslope of covered disposal sites to collect and transport sediment-laden flow to sediment traps or basins. Ditches should be left in place until the disposal site is sealed and stabilized with cover vegetation.

Diversions are permanent or temporary shallow drainageways excavated along the contour of graded slopes and having a supporting earthen ridge (dike or berm) constructed along the downhill edge of the drainageway. Essentially,

FIGURE 3-7

TYPICAL DRAINAGE DITCH AT BASE OF DISPOSAL SITE

(Source: JRB and Emcon Associates, 1980)



a diversion is a combination of a ditch and a dike (EPA, 1976). Diversions are used primarily to provide more permanent erosion control on long slopes subject to heavy flow concentrations. They may be constructed across long slopes to divide the slope into nonerosive segments. Diversions may also be constructed at the top or at the base of long graded slopes at disposal sites to intercept and carry flow at nonerosive velocities to natural or prepared outlets. Diversions are recommended for use only in slopes of 15 percent or less (EPA, 1976).

Grassed waterways (or channels) are graded drainageways that serve as outlets for diversions or berms. Waterways are stabilized with suitable vegetation and are generally designed to be wide and shallow in order to convey runoff down slopes at nonerosive velocities. Waterways may be constructed along the perimeter of disposal sites located within natural slopes, or they may be constructed as part of the final grading design for disposal areas that have been capped and revegetated.

3.4.2.2. Design and Construction Considerations

Ditches, diversions, and waterways are generally of V-shaped, trapezoidal or parabolic cross-section design. The specific design will be dependent on local drainage patterns, soil permeability, annual precipitation, area land use, and other pertinent characteristics of the contributing watershed. In general, such drainageways should be designed to accommodate flows resulting from rainfall events (storms) of 10- or 25-year frequency. More importantly, they should be designed and constructed to intercept and convey such flows at non-erosive velocities.

Figure 3-8 depicts the effect of drainage channel shape on relative velocity of conveyed flows. In general, the wider and shallower the channel cross-section, the less the velocity of contained flow and therefore, the less the potential for erosion of drainageway side slopes. Where local conditions dictate the necessity of building narrower and deeper channels, or where slopes are steep and flow velocities are excessive, the channel will require stabilization through seeding and mulching or the use of stone riprap to line channel bottoms and break up flow.

Table 3-9, below, presents maximum permissible design velocities for flow in ditches and grassed waterways, based on the channel grade and stabilizing cover material.

Figure 3-9 shows the standard design for drainage ditches. These structures are designed for short-term application only, for upslope drainage areas of less than 5 acres. A minimum grade of one percent, draining to a stabilized outlet such as a grassed waterway or, where necessary, to a sediment basin or trap, is recommended for temporary ditches. For channel slopes greater than 5 percent, stabilization with grasses, mulches, sod, or stone riprap will be necessary. As with all temporary structures, periodic inspection and maintenance are required to ensure structural integrity and effective performance (EPA, 1976).

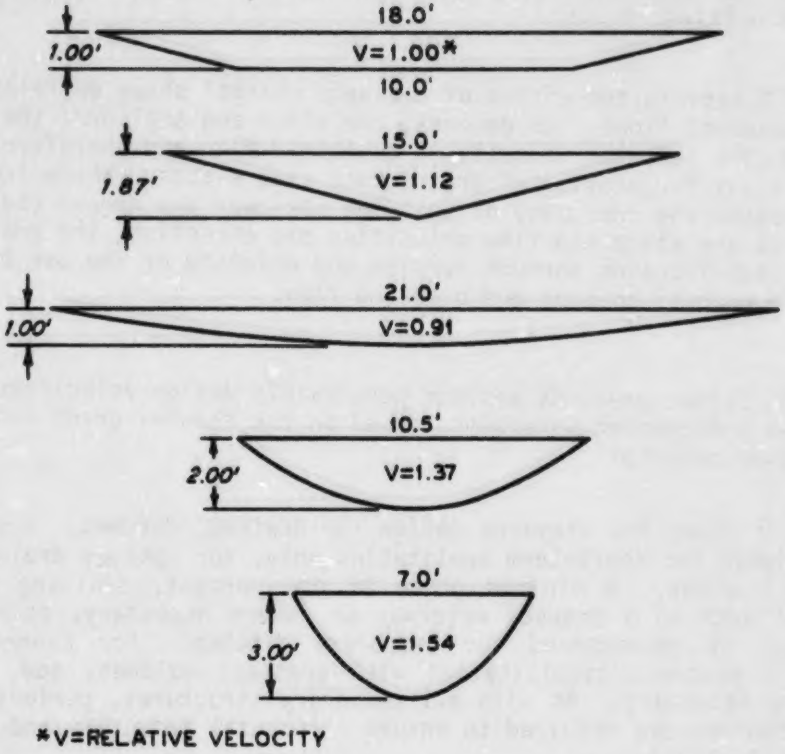
Figure 3-10 presents general design features of parabolic and trapezoidal diversions. A formal design is not required for diversions used as temporary water-handling structures. General design and construction criteria for permanent diversions include the following: (EPA, 1976).

- Diversion location is determined on the basis of outlet conditions, topography, soil type, slope length and grade.
- Constructed diversion shall have capacity to carry peak discharge from 25-year design storm.

FIGURE 3-8

EFFECT OF DRAINAGE DITCH ON VELOCITY

(Source: Lutton, et al., 1979)


18.0'
1.00'
V=1.00*
10.0'
15.0'
1.87'
V=1.12
21.0'
1.00'
V=0.91
10.5'
2.00'
V=1.37
7.0'
3.00'
V=1.54
*V=RELATIVE VELOCITY

- The maximum grade of the diversion may be determined by using design velocity of the flow based on stabilization by cover type (Table 3-9, above).

- The diversion channel shall be parabolic or trapezoidal in shape, with side slopes no steeper than 2:1.

- The supporting ridge (dike or berm) shall have a minimum width of 4 ft.; freeboard shall be 0.3 ft. minimum over peak water level in channel.

- Each diversion shall have a stable outlet such as a natural waterway, stabilized open channel, chute, or downpipe.

- Stabilization: For design velocities 3.5 ft/sec, seeding and mulching for vegetative establishment; for velocities 3.5 ft/sec, stabilize with sod or with seeding protected by jute or excelsior matting.

- For channels that carry flow during dry weather (base flow) due to groundwater discharge or delayed subsurface runoff the bottom should be protected with a stone center as shown in Figure 3-11 for grassed waterways; subsurface drainage with gravel/stone trenches may be required where the water table is at or near the surface of the channel bottom.

- Fills shall be compacted as needed to prevent unequal settlement.

- All trees, bushes, stumps, and obstructions shall be cleared to prevent improper functioning of the channel.

Figures 3-11 and 3-12 depict general design configurations for grassed waterways. The design and construction criteria presented above for diversions are applicable to grassed waterways also.

3.4.2.3 Advantages and Disadvantages

When they are carefully designed, constructed, and maintained, ditches, diversions, and grassed waterways will control surface erosion and infiltration at disposal sites by intercepting and safely diverting storm runoff to downslope or off-site outlets. When situated at the base of disposal site slopes, they function to protect off-site habitat from possible contamination by sediment-laden runoff. These structures are generally constructed of readily available fill, by well-established techniques.

Temporary ditches and diversions, however, entail added costs because they require inspections and maintenance. Grassed waterways must be periodically mowed to prevent excessive retardation of flow and subsequent ponding of water. Also, periodic resodding, remulching, and fertilizing may be required to maintain vegetated channels.

TABLE 3-9

PERMISSIBLE DESIGN VELOCITIES FOR STABILIZED DIVERSIONS AND WATERWAYS

Cover	Channel grade (%)	Maximum design velocity (ft/sec)
Vegetative		
● Bermuda grass	0-5	6
	5-10	5
	10	4
● Reed canary grass; Tall fescue; Kentucky bluegrass	0-5	5
	5-10	4
	10	3
● Grass-legume mix	0-5	4
	5-10	3
● Red fescue; Redtop, sericea lespedeza	0-5	2.5
● Annuals; Small grain (rye, oats, barley); Ryegrass	0-5	2.5

(Source: EPA, 1976)

If fertilization is used, an additional disadvantage is introduced, in that nitrogen and phosphorus are added to drainage wastes, which then contribute to the problem of accelerated eutrophication in receiving water bodies.

It may also be necessary to install temporary straw-bale check dams, staked down at 50- to 100-foot intervals, across ditches and waterways in order to prevent gulley erosion and to allow vegetative establishment (Tour-

FIGURE 3-9

STANDARD DESIGN FOR DRAINAGE DITCHES

(Source: U.S. EPA, 1976)

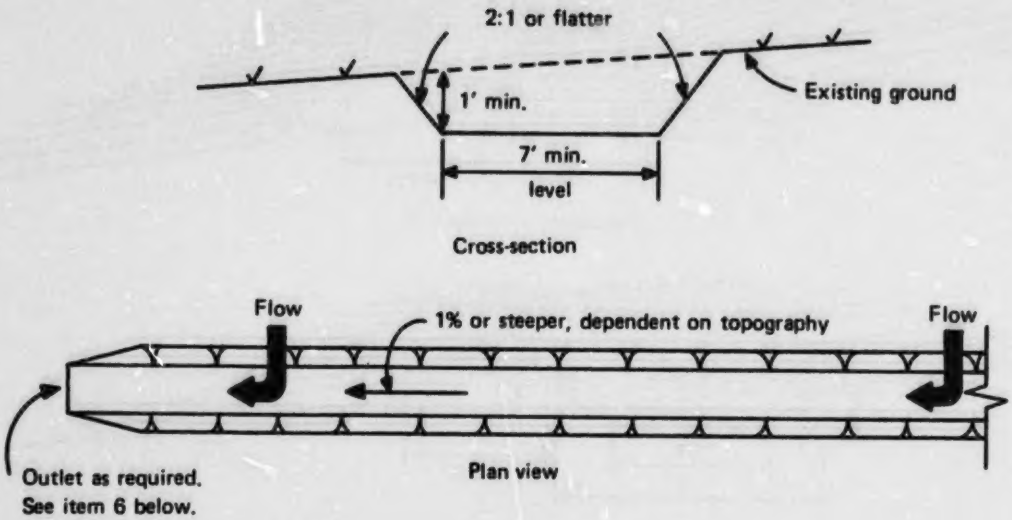
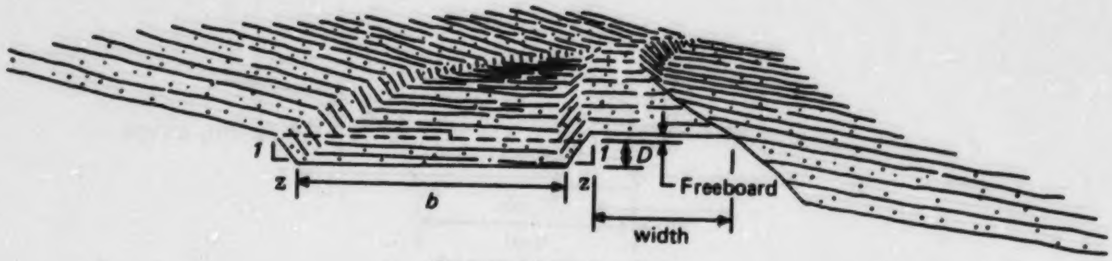

2:1 or flatter
1' min.
Existing ground
7' min.
level
Cross-section
Flow
1% or steeper, dependent on topography
Flow
Outlet as required.
See item 6 below.
Plan view

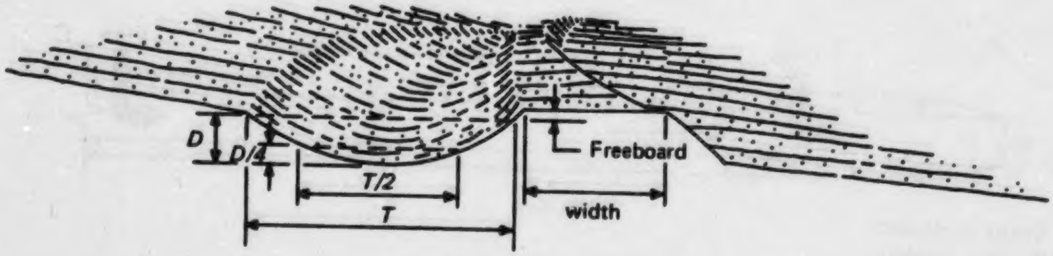
FIGURE 3-10

GENERAL DESIGN FEATURES OF DIVERSIONS

(Source: U.S. EPA, 1976)


1
z
b
1
z
D
Freeboard
width

Trapezoidal cross-section

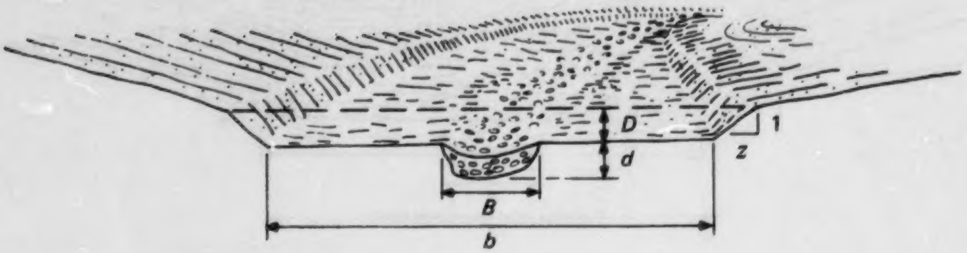

D
D/4
T/2
T
Freeboard
width

Parabolic cross-section

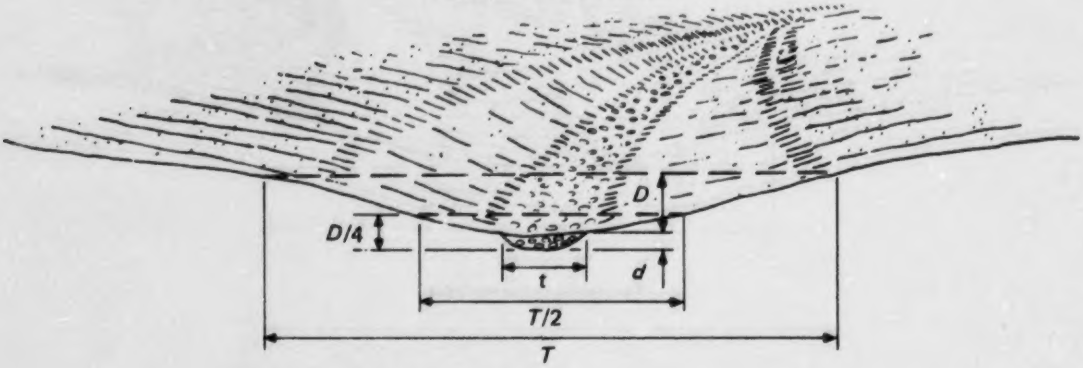
FIGURE 3-11

GRASSED WATERWAYS WITH STONE CENTERS

(Source: U.S EPA, 1976)



Trapezoidal cross-section



Parabolic cross-section

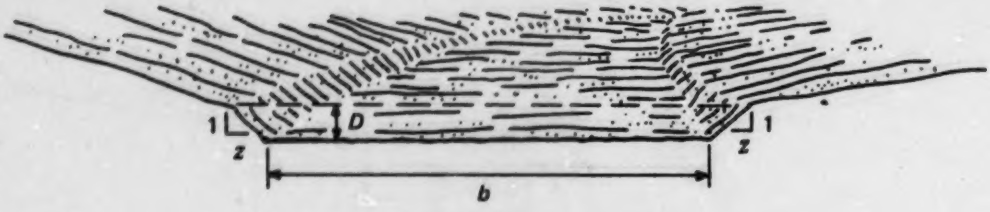
bier and Westmacott, 1974). The installation and ultimate removal of these check dams will add to the costs associated with diversions and waterways.

Permanent diversions and waterways are more cost-effective techniques than temporary structures for controlling erosion and infiltration on a long-term basis at inactive disposal sites.

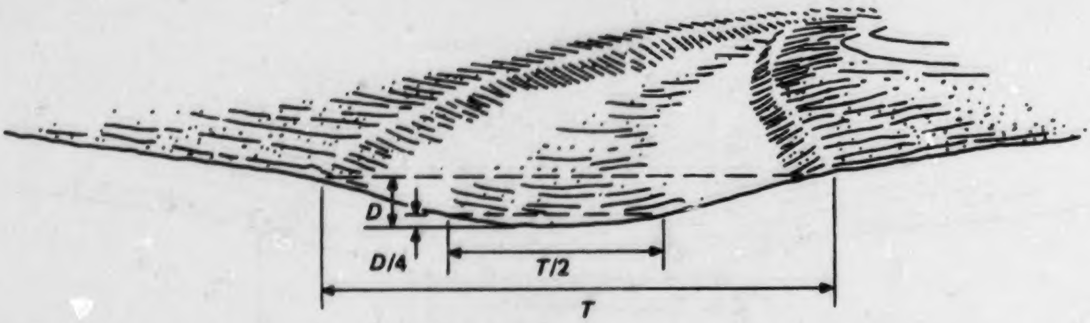
FIGURE 3-12

GRASSED WATERWAYS

(Source: U.S. EPA, 1976



Trapezoidal cross-section



Parabolic cross-section

3.4.2.4 Costs

As with all surface-water diversion structures, costs for ditches, diversions, and waterways will be highly variable because of site-specific differences in fill materials and equipment availability, design capacity and life expectancy of the structure, etc. Unit costs associated with the construction and maintenance of these structures are presented at the end of this chapter in Table 3-15.

3.4.3 Terraces and Benches

3.4.3.1 Description and Applications

Terraces and benches are relatively flat areas constructed along the contour of very long or very steep slopes to slow down runoff and divert it into ditches or diversions for off-site transport at nonerosive velocities. These structures are also known as bench terraces or drainage benches.

Although benches and terraces are slope reduction devices, they are generally constructed with reverse fall or natural fall (see Figure 3-13), to divert water to stabilized drainageways. Benches and terraces may be used to break up steeply graded slopes of covered disposal sites into less erodible segments. Upslope of disposal sites, they act to slow and divert storm runoff around the site. Downslope of landfill areas, they act to intercept and divert sediment-laden runoff to traps or basins. Hence, they may function to hydrologically isolate active disposal sites, to control erosion of cover materials on completed fills, or to collect contaminated sediments eroded from disposal areas. For disposal sites undergoing final grading (after capping and prior to revegetation), construction of benches or terraces may be included as part of the integrated site closure plan.

3.4.3.2 Design and Construction Considerations

Benches and terraces generally do not require a formal design plan. Figure 3-14 presents the design for a typical drainage bench located on the slope of a covered landfill. This particular bench is designed with a natural fall. It is intended for long-term erosion protection as the associated V-shaped channel is asphalt concrete-lined. Diversions and ditches included in bench/terrace construction may be seeded and mulched, sodded, stabilized with riprap or soil additives, or stabilized by any combination of these methods. Lining the channels with concrete or grouted riprap is a more costly alternative.

The width and spacing between benches and terraces will depend on slope steepness, soil type, and slope length. In general, the longer and steeper the slope and the more erodible the cover soil, the less the distance between drainage benches should be. This will maximize the erosion reduction afforded by constructed benches. For slopes greater than 10 percent in steepness, the maximum distance between drainage benches should be approximately 100 feet, i.e., a bench every 10 feet of rise in elevation (EPA, 1976).

When the slope is greater than 20 percent, it has been recommended that benches be placed every 20 feet of rise in elevation (EPA, 1979). Benches

FIGURE 3-13A

BENCH TERRACES WITH RESERVE FALL

(Adapted from Tourbier and Westmacott, 1974)

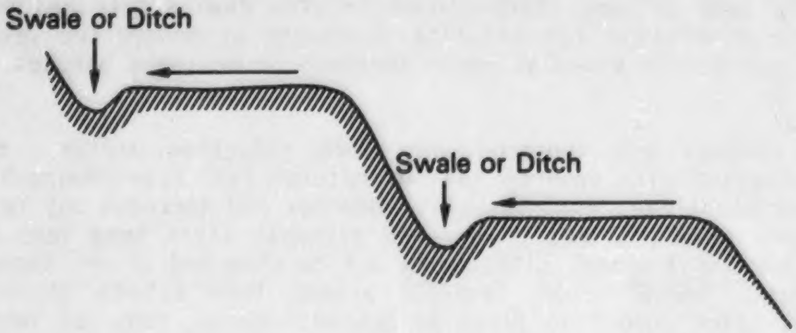

Swale or Ditch
Swale or Ditch

FIGURE 3-13B

BENCH TERRACES WITH NATURAL FALL

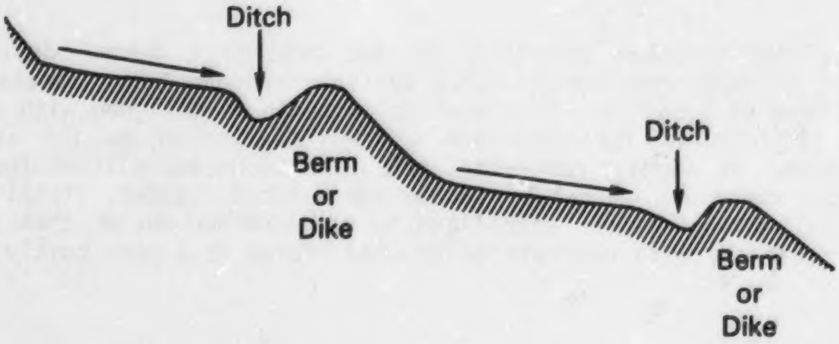
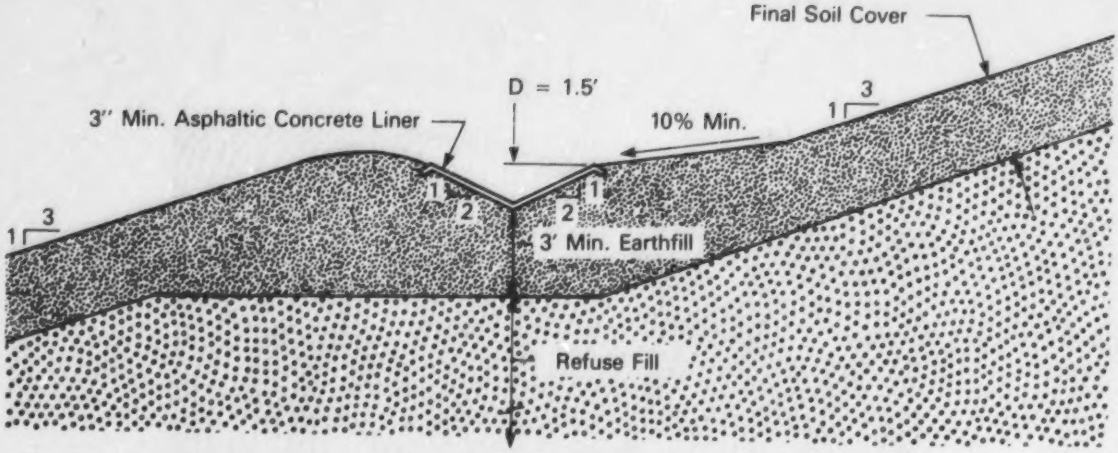

Ditch
Berm
or
Dike
Ditch
Berm
or
Dike

FIGURE 3-14

TYPICAL DRAINAGE BENCH



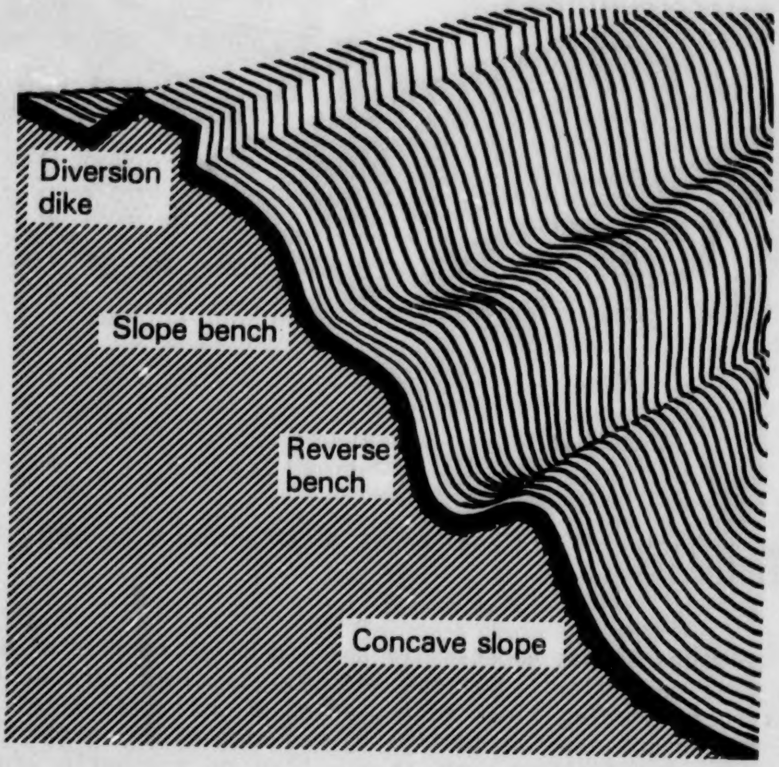
should be of sufficient width and height to withstand a 24-hour, 25-year storm (EPA, 1979).

Bench terraces do not necessarily have to be designed with diversions or ditches to intercept flow. Reverse benches and slope benches (Figure 3-15) may be constructed during final site grading on well-stabilized slopes (e.g., vegetated) to enhance erosion control by reducing slope length and steepness. At sites where an effective cap (e.g., clay or synthetic liner) has been constructed, or for sites located in arid regions, these nondrainage benches will function to slow sheet runoff and allow greater infiltration rates, which will aid in the establishment of a suitable vegetative cover. For most disposal sites in wet climates, however, where leachate generation and cover erosion are major problems, benches and terraces should be designed in association with drainage channels that intercept and transport heavy, concentrated surface flows safely off-site.

FIGURE 3-15

SLOPE REDUCTION MEASURES

(Source: U.S. EPA, 1976)


Diversion dike
Slope bench
Reverse bench
Concave slope

As with other earthen erosion control structures, benches and terraces should be sufficiently compacted and stabilized with appropriate cover (grasses, mulches, sod) to accommodate local topography and climate. They should be inspected during or after major storms to ensure proper functioning and structural integrity. If bench slopes become badly eroded or if their surfaces become susceptible to ponding from differential settlement, regrading and sodding may be necessary.

3.4.3.3. Advantages and Disadvantages

In areas of high precipitation, drainage benches and terraces are proven effective in reducing velocity of storm runoff and thereby controlling erosion. For excessively long and steep slopes above, on, or below disposal sites, these structures are cost-effective methods for slowing and diverting runoff. They may also be used to manage downslope washout of disposal site sediments that may be contaminated with hazardous waste components. Terraces and benches are easily incorporated into final grading schemes for disposal sites, and do not require special equipment or materials for their construction.

If improperly designed or constructed, bench terraces will not perform efficiently and may entail excessive maintenance and repair costs. It is important that these structures be stabilized with vegetation as soon as possible after grading and compaction, or they may become badly eroded and require future resodding or chemical stabilization. Benches and terraces also require periodic inspections, especially after major rainfall events.

3.4.3.4 Costs

The costs of bench and terrace construction will depend on the amount of fill required, the local availability of fill materials and grading equipment, the size and type of diversion channels to be installed, and the local costs of seeding, mulching, and other stabilizing materials. The frequency and extent of required maintenance will add to these costs.

Unit costs for these construction and maintenance activities are presented in Table 3-15 at the end of this chapter.

3.4.4 Chutes and Downpipes

3.4.4.1 Description and Applications

Chutes and downpipes are temporary structures used to carry concentrated flows of surface runoff from one level to a lower level without erosive damage. They generally extend downslope from earthen embankments (dikes or berms) and convey water to stabilized outlets located at the base of terraced slopes.

Chutes (or flumes) are open channels, normally lined with bituminous concrete, Portland cement concrete, grouted riprap, or similar nonerodible material. Temporary paved chutes are designed to handle concentrated surface flows from drainage benches located near the base of the long, steep slopes at disposal sites.

Downpipes (downdrains; pipe slope drains) are temporary structures constructed of rigid piping (such as corrugated metal) or flexible tubing of heavy-duty fabric. They are installed with standard prefab-ricated entrance sections and are designed to handle flow from drain-age areas of 5 acres or less. Like paved chutes, downpipes discharge to stabilized outlets or sediment traps. Downpipes may be used to collect and transport runoff from long, isolated outslopes or from small disposal areas located along steep slopes.

Temporary downpipes may also be constructed by joining half-round sections of bituminous fiber or concrete pipe (Figure 3-16). These structures may be quickly constructed for emergency situations when downslope ditches or waterways overflow during severe storms and threaten to erode the base of disposal fill areas.

3.4.4.2 Design and Construction Considerations

Chutes and downpipes are temporary structures that do not require formal design. General design criteria are presented in Figures 3-17 (paved chute), 3-18 (rigid downpipe), and 3-19 (flexible downpipes).

Chutes are designed to handle flows based on two basic size groups. Paved chutes of size group A have the following three qualifications:

- Height (H) of dike at entrance = 1.5 feet minimum
- Depth (D) of chute down the slope = 8 inches minimum
- Length (L) of inlet/outlet sections = 5 feet minimum

Similarly, chutes of size group B meet the following criteria:

- H = 2 feet minimum
- D = 10 inches minimum
- L = 6 feet minimum

FIGURE 3-16

HALF-ROUND BITUMINOUS FIBER PIPE USED FOR TEMPORARY HANDLING OF CONCENTRATED FLOW

(Source: U.S. EPA, 1976)

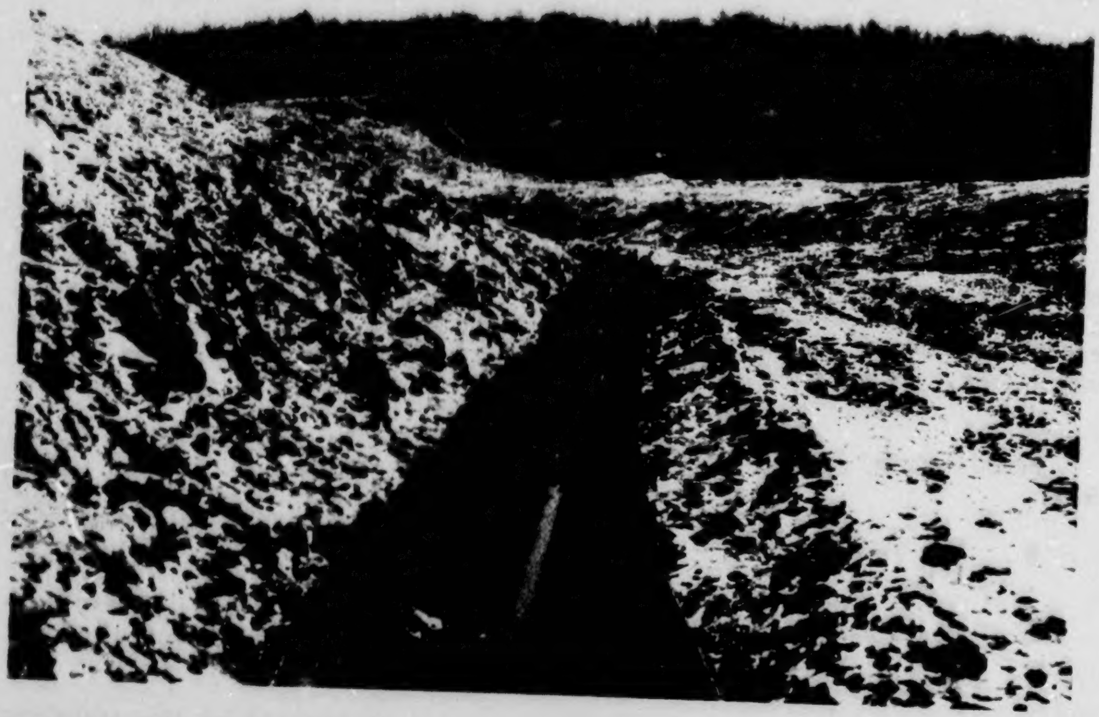


Table 3-10 below presents the bottom width and maximum drainage area for designed chutes of the two size groups.

TABLE 3-10

BOTTOM WIDTHS AND MAXIMUM DRAINAGE AREAS FOR TEMPORARY CHUTES

Size group	Bottom width, D (ft.)[1]	Maximum drainage area (acres)	Size group	Bottom width, D (ft.)[1]	Maximum drainage area (acres)
A-2	2	5	B-4	4	14
A-4	4	8	B-6	6	20
A-6	6	11	B-8	8	25
A-8	8	14	B-10	10	31
A-10	10	18	B-12	12	36

[1] Source: U.S. EPA, 1976.

If 75 percent or more of the drainage area has good vegetative cover (established grasses and/or shrubs) throughout the design life of the chute, the drainage areas listed in Table 3-10 may be increased by 50 percent. If 75 percent or more of the drained area has a mulch cover throughout the structure's life, the areas may be increased by 25 percent (EPA, 1976).

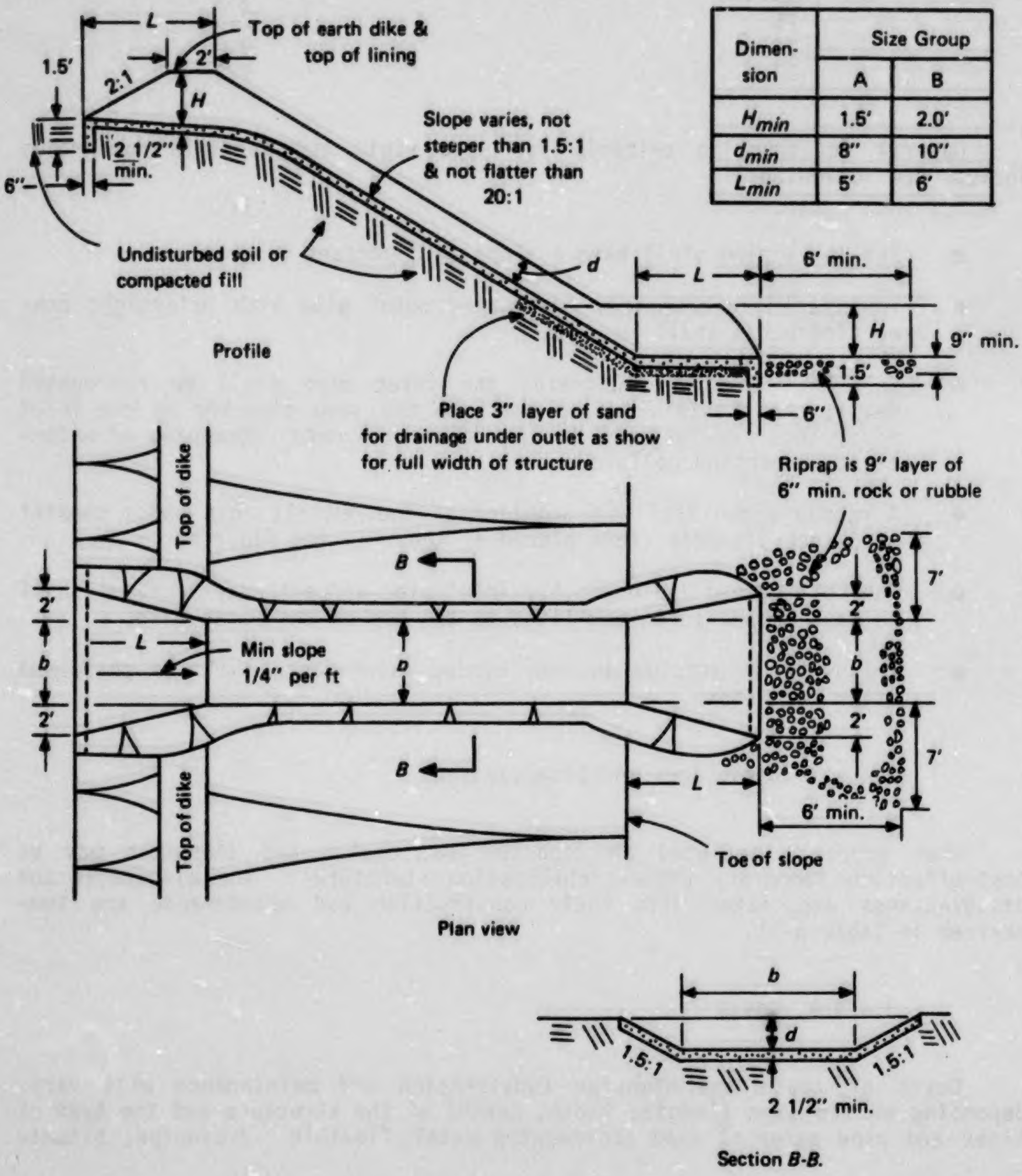
Paved chute construction considerations include the following (EPA, 1976):

- The structure shall be placed on undisturbed soil or well-compacted fill.
- The lining shall be placed by beginning at the lower end and proceeding upslope; the lining shall be well-compacted, free of voids, and reasonably smooth.
- The cut-off walls at the entrance and at the end of the asphalted discharge aprons shall be continuous with the lining.
- An energy dissipator (riprap bed) shall be used to prevent erosion at the outlet.

FIGURE 3-17

PAVED CHUTE (OR FLUME)

(Source: U.S. EPA, 1976)



Dimension	Size Group	
	A	B
H_{min}	1.5′	2.0′
d_{min}	8″	10″
L_{min}	5′	6′

L
Top of earth dike & top of lining
1.5′
2′
2:1
H
2 1/2″ min.
6″
Slope varies, not steeper than 1.5:1 & not flatter than 20:1
Undisturbed soil or compacted fill
d
Profile
L
6′ min.
H
9″ min.
1.5′
6″
Place 3″ layer of sand for drainage under outlet as show for full width of structure
Riprap is 9″ layer of 6″ min. rock or rubble
Top of dike
B
2′
L
Min slope 1/4″ per ft
b
b
2′
B
7′
2′
b
2′
7′
L
6′ min.
Top of dike
Toe of slope
Plan view
b
d
1.5:1
1.5:1
2 1/2″ min.
Section B-B.

For downpipes (Figure 3-18 and 3-19), the maximum drainage area is determined from the diameter of the piping, as follows (EPA, 1976):

PIPE/TUBING DIAMETER, D (INCHES)	MAXIMUM DRAINAGE AREA (ACRES)
12	.5
18	1.5
21	2.5
24	3.5
30	5.0

General construction criteria for both rigid and flexible downdrains include the following:

- The inlet pipe shall have a slope of 3 percent or greater.
- For the rigid downpipe, corrugated metal pipe with watertight connecting bands shall be used.
- For the flexible downdrain, the inlet pipe shall be corrugated metal; the flexible tubing shall be the same diameter as the inlet pipe, securely fastened to the inlet with metal strapping or watertight connecting collars.
- A riprap apron shall be provided at the outlet; this shall consist of 6-inch diameter stone placed as shown in the figures.
- The soil around and under the inlet pipe and entrance sections shall be hand-tamped in 4-inch lifts to the top of the earth dike.
- Follow-up inspection and any needed maintenance shall be performed after each storm.

3.4.4.3 Advantages and Disadvantages

When properly designed and constructed, chutes and downpipes may be cost-effective temporary grade-stabilization structures. The advantages and disadvantages associated with their construction and maintenance are summarized in Table 3-11.

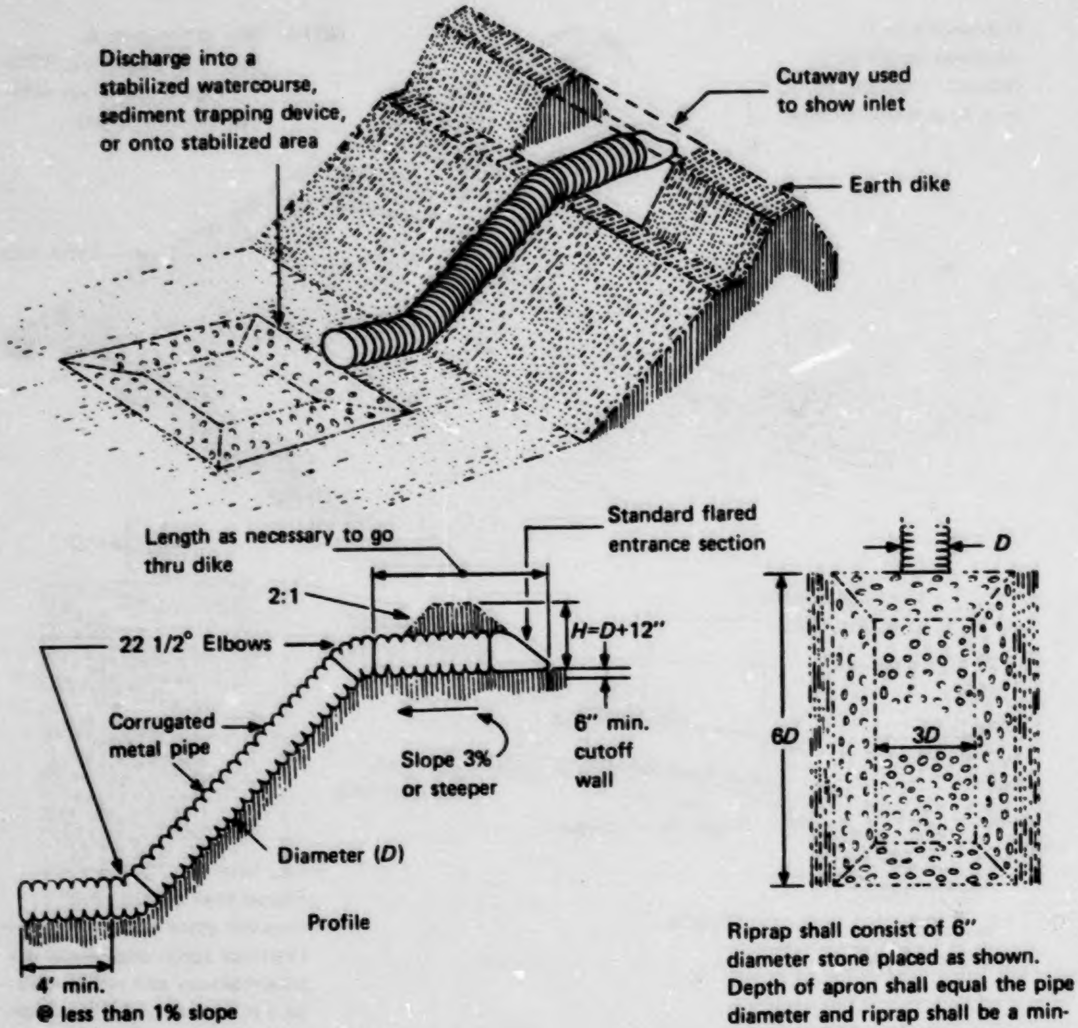
3.4.4.4 Costs

Costs of chute and downpipe construction and maintenance will vary, depending on the size (length, width, depth) of the structure and the type of liner and pipe material used (corrugated metal, flexible drainpipe, bitumi-

FIGURE 3-18

RIGID DOWNPIPE

(Source: U.S. EPA, 1976)


Discharge into a stabilized watercourse, sediment trapping device, or onto stabilized area
Cutaway used to show inlet
Earth dike
Length as necessary to go thru dike
Standard flared entrance section
2:1
22 1/2° Elbows
H=D+12"
Corrugated metal pipe
Slope 3% or steeper
6" min. cutoff wall
Diameter (D)
4' min.
@ less than 1% slope
D
6D
3D

Profile

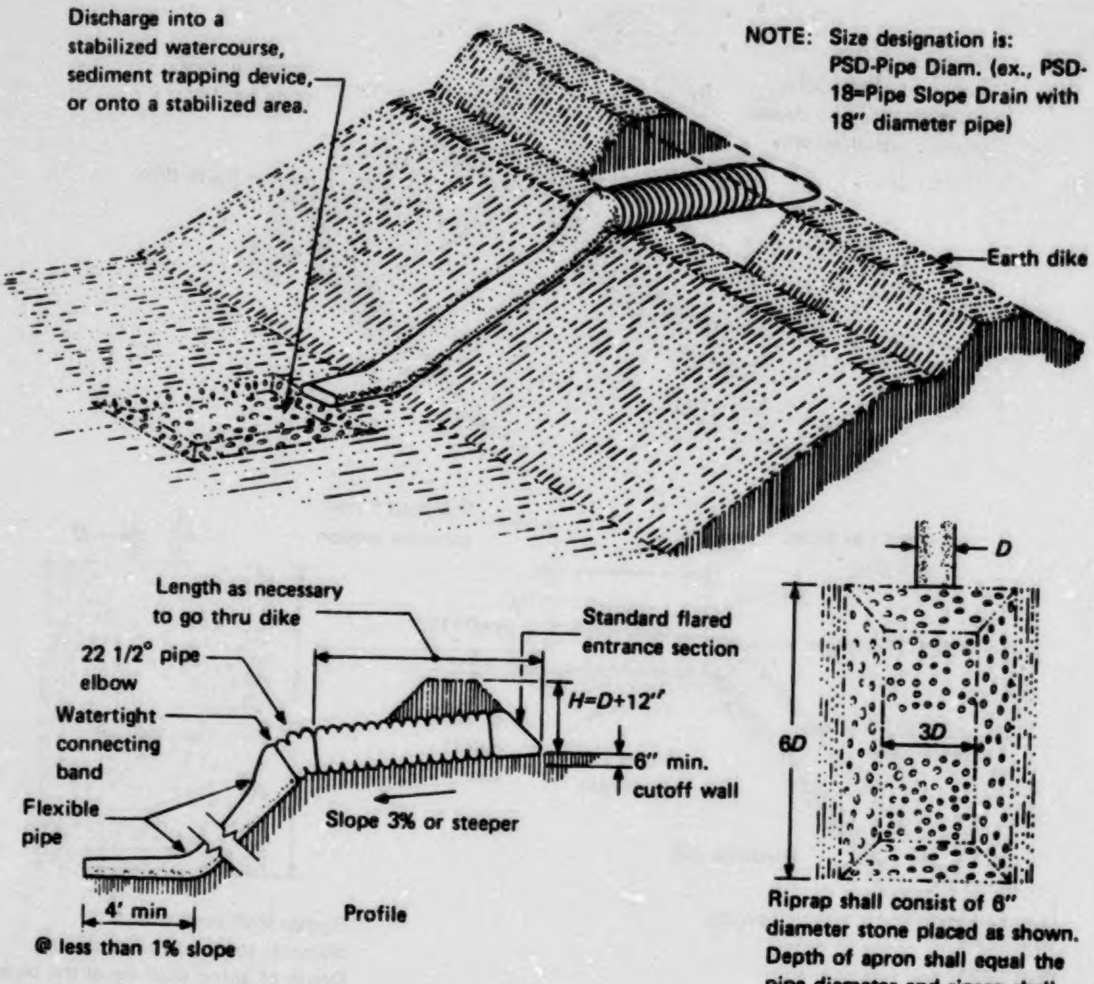
Riprap shall consist of 6" diameter stone placed as shown. Depth of apron shall equal the pipe diameter and riprap shall be a minimum of 12" in thickness.

Riprap apron plan

FIGURE 3-19

FLEXIBLE DOWNDRAIN

(Source: U.S. EPA, 1976)


Discharge into a stabilized watercourse, sediment trapping device, or onto a stabilized area.
NOTE: Size designation is: PSD-Pipe Diam. (ex., PSD-18=Pipe Slope Drain with 18" diameter pipe)
Earth dike
Length as necessary to go thru dike
Standard flared entrance section
22 1/2° pipe elbow
H=D+12"
Watertight connecting band
6" min. cutoff wall
Flexible pipe
Slope 3% or steeper
4' min
@ less than 1% slope
Profile
D
6D
3D

Riprap shall consist of 6" diameter stone placed as shown. Depth of apron shall equal the pipe diameter and riprap shall be a minimum of 12" in thickness.

Riprap apron plan

TABLE 3-11

SUMMARY OF ADVANTAGES AND DISADVANTAGES OF CHUTES AND DOWNPIPES

Advantages	Disadvantages
● Construction methods are inexpensive and quick; suitable for emergency measures.	● Provide only temporary erosion control while slopes are stabilized with vegetative growth.
● No special materials or equipment are required.	● Entail extra cost for periodic inspections and maintenance and ultimate removal.
● Are effective in preventing erosion on long, steep slopes.	● If improperly designed, may overflow and cause severe erosion in conconcentrated areas.
● Can be used to channel storm runoff to sediment traps, drainage basins, or stabilized waterways for off-site transport.	● Downpipes are suitable for drainage areas 5 acres in size; limited applications in general.
● Can be key element in combined surface control systems.	

nous fiber, PVC). Unit costs associated with these temporary structures are presented at the end of the chapter in Table 3-15.

3.4.5 Levees

3.4.5.1 Description and Applications

Levees are earthen embankments that function as flood protection structures in areas subject to inundation from tidal flow or riverine flooding. Levees create a barrier to confine floodwaters to a floodway and to protect structures behind the barrier. They are most suitable for installation in flood fringe areas or areas subject to storm tide flooding, but not for areas directly within open floodways (Tourbier and Westmacott, 1974).

Flood containment levees may be constructed as perimeter embankments surrounding disposal sites located in floodplain fringe areas, or they may be installed at the base of landfills along slope faces that are subject to periodic inundation. Levees serve to protect land disposal sites from floodwaters, which may erode cover materials and transport waste materials offsite, or which may add water to waste materials and thus increase hazardous leachate production.

Levees are generally constructed of compacted impervious fill. Special drainage structures are often required to drain the area behind the embankment. Levees are normally constructed for long-term flood protection, but they require periodic inspection and maintenance to assure proper functioning. They may be costly to build and maintain, but if properly designed on a site-specific basis, levees will reduce flooding hazards at critical waste disposal areas.

3.4.5.2 Design and Construction Considerations

To provide adequate flood protection, levees should be constructed to a height capable of containing a design flood of 100-year magnitude. Levees designed to protect disposal areas from maximum flood levels of lesser magnitude/greater frequency (e.g., 50-year flood) may not provide sufficient flood protection, particularly for sites known to contain large quantities of hazardous wastes. Elevation of 100-year base flood crests can be determined from floodplain analyses typically performed by state or local flood control agencies. A minimum levee elevation of 2 feet above the 100-year flood level is recommended (JRB Associates and Emcon Associates, 1980).

Figure 3-20 presents design features of a typical levee constructed at the toe of a landfill slope. This design is appropriate for new or uncompleted disposal sites; filled wastes may eventually be placed on the inboard slope of the levee. Where levee construction is impractical due to lack of soil or limited space, perimeter protection of vulnerable landfill slopes may be afforded by the design shown in Figure 3-21. A minimum top width of 10 feet is recommended for most levees; this will permit easy access for construction and maintenance equipment (Linsley and Franzini, 1979).

Ideal construction of levees is with erosion-resistant, low-permeability soils, preferably clay. Most levees are homogeneous embankments; but if impermeable fill is lacking, or if seepage through and below the levee is a problem, then construction of a compacted impervious core or sheet-pile cutoff extending below the levee to bedrock (or other impervious stratum) may be

FIGURE 3-20

TYPICAL LEVEE AT BASE OF DISPOSAL SITE

(Source: JRB Associates and Emcon Associates, 1980)

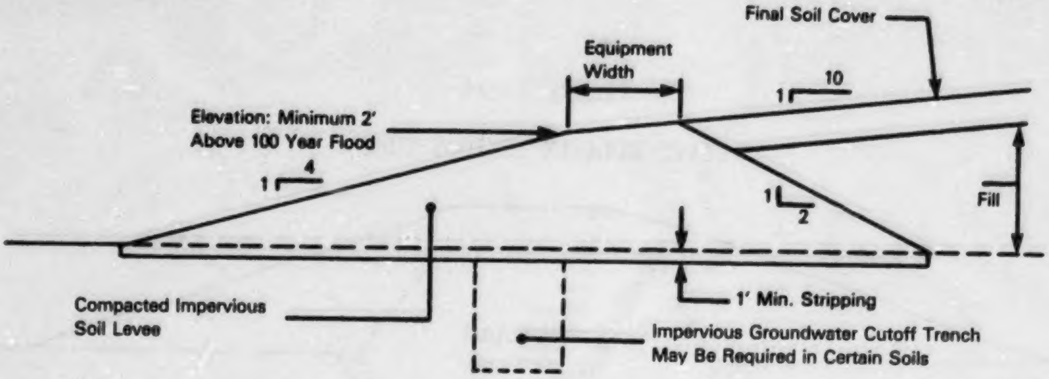
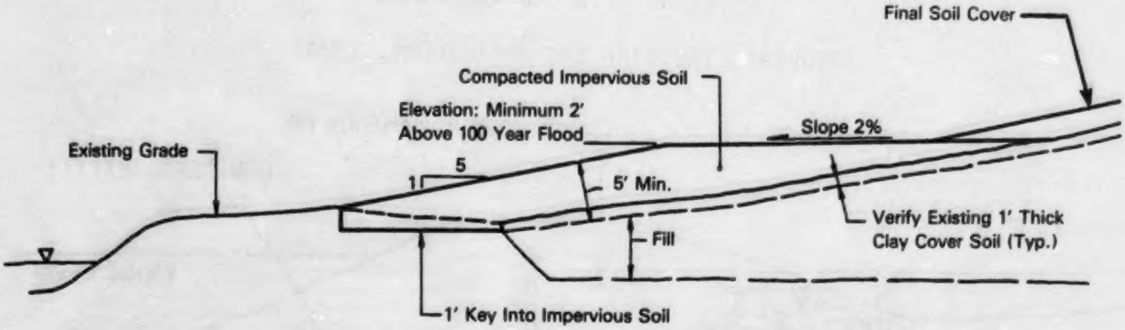

Final Soil Cover
Equipment Width
1 10
Elevation: Minimum 2' Above 100 Year Flood
1 4
1 2
Fill
Compacted Impervious Soil Levee
1' Min. Stripping
Impervious Groundwater Cutoff Trench May Be Required in Certain Soils

FIGURE 3-21

PERIMETER FLOOD PROTECTION STRUCTURE

(Source: JRB Associates and Emcon Associates, 1980)


Final Soil Cover
Compacted Impervious Soil
Elevation: Minimum 2' Above 100 Year Flood
Slope 2%
Existing Grade
1 5
5' Min.
Fill
Verify Existing 1' Thick Clay Cover Soil (Typ.)
1' Key Into Impervious Soil

necessary. Figure 3-22 depicts these two special cases. Excess seepage through the levees should be collected with gravel-filled trenches or tile drains along the interior of the levee. After draining to sumps, the seepage can be pumped out over the leeve. Levee bank slopes, especially those constructed of less desirable soils (silt, sands), should be protected against erosion by sodding, planting of shrubs and trees, or use of stone riprap (Linsley and Franzini, 1979).

FIGURE 3-22A

LEVEE WITH IMPERVIOUS CORE

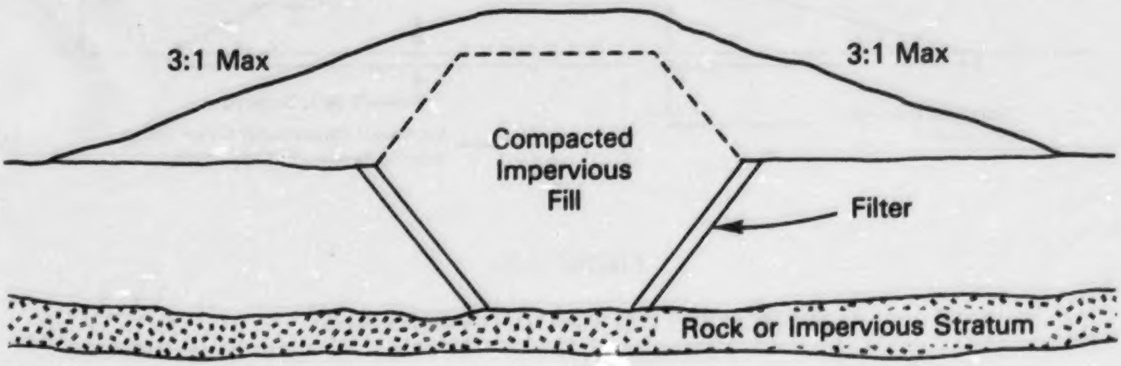
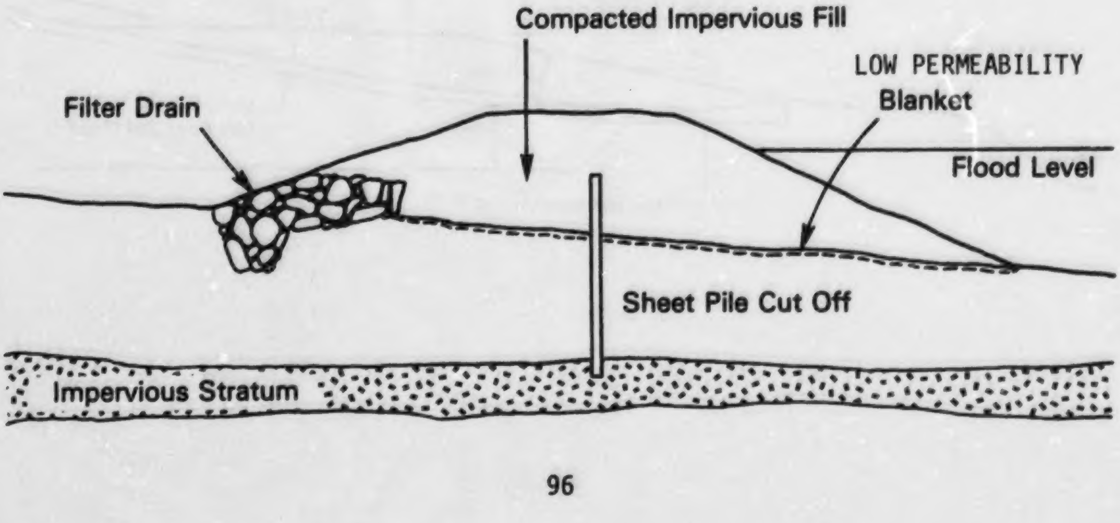

3:1 Max
3:1 Max
Compacted Impervious Fill
Filter
Rock or Impervious Stratum

FIGURE 3-22B

LEVEE WITH CUTOFF AND DRAIN

(Source: Tourbier and Westmacott, 1974)


Compacted Impervious Fill
LOW PERMEABILITY Blanket
Filter Drain
Flood Level
Sheet Pile Cut Off
Impervious Stratum

Fill material used in levee construction should be compacted in layers, with the least pervious layer along the riverside of the levee. Because the use of levees will reduce floodplain storage capacity, fill material should be dredged from borrow pits within the floodplain to provide alternate storage volume for floodwaters. This measure will help control rising floodpeaks and prevent an increase in depth of downstream flood stage (Tourbier and Westmacott, 1974).

Storm runoff from precipitation falling on the drainage area behind the levee may cause backwater flooding. To handle such interior drainage, upslope interceptor ditches, diversions, or grassed waterways can be used to channel runoff to downslope holding basins (for subsequent pumping) or to off-site streams for natural gravity discharge. Another method to handle backwater flow is the installation of pressure conduits (with upslope intake works) that discharge beneath the levee. These conduits should be equipped with tidal gates or backwater valves to prevent back-flow and regulate discharge.

Because of the relatively long, flat side slopes of levees, an embankment of any considerable height requires a very large base width. For locations with limited space and fill material, or excessive real estate costs, the use of concrete floodwalls is preferred as an alternative to levee construction. Floodwalls are designed to withstand the hydrostatic pressure exerted by water at the design flood level. They are subject to flood loading on one side only; consequently, they need to be well founded (Tourbier and Westmacott, 1974). Figure 3-23 presents typical floodwall sections. Like levees, floodwalls may require sub-surface cutoffs and interior drainage structures to handle excessive seepage or backwater flow.

3.4.5.3 Advantages and Disadvantages

The advantages and disadvantages associated with flood protection levees at waste disposal sites are summarized in Table 3-12.

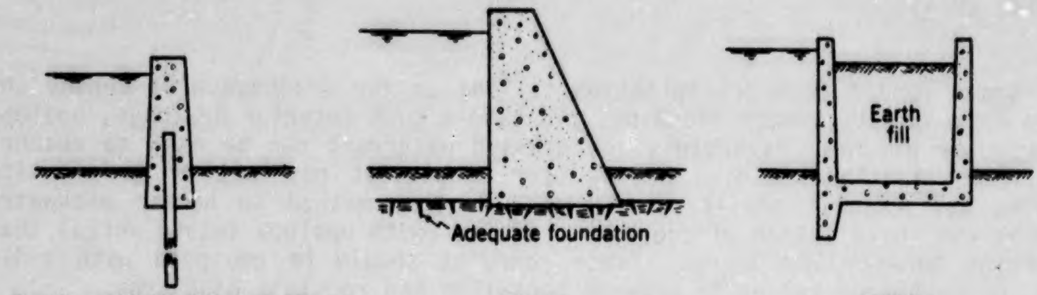
3.4.5.4 Costs

Costs associated with constructing and maintaining levees will depend on site-specific design variables, availability of suitable embankment soil, and the local frequency and magnitude of flooding. If backwater flooding or seepage is a problem, then special structures must be included in the construction plan. Regular annual inspection for evidence of bank caving, bank sloughing, erosion, and foundation settlement will also increase associated costs.

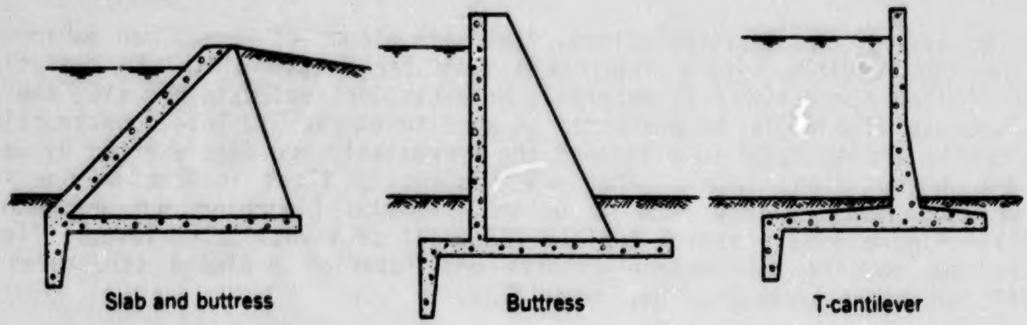
FIGURE 3-23

SOME TYPICAL FLOODWALL SECTIONS

(Source: Linsley and Franzini, 1979)



Simple cantilever and sheet pile Gravity Cellular



Slab and buttress Buttress T-cantilever

Unit costs relating to levee construction and maintenance are included in Table 3-15 in Section 3.4.8.

3.4.6 Seepage Basins and Ditches

3.4.6.1 General Description and Applications

Seepage or recharge basins are designed to intercept runoff and recharge the water downgradient from the site so that groundwater contamination and leachate problems are avoided or minimized.

3.4.6.2 Design and Construction Considerations

There is considerable flexibility in the design of seepage basins and ditches. Figures 3-24 through 3-27 illustrate possible design variations.

TABLE 3-12

SUMMARY OF EVALUATION OF LEVEES

Advantages	Disadvantages
● Can be built at relatively low cost from materials available at site	● Flooding from storm runoff behind levee may be a problem
● Will provide long-term flood protection if properly designed and constructed	● Loss of flow storage capacity, with greater potential of downstream flooding
● Controls major erosive losses of waste and cover material; prevents massive leachate production and subsequent contamination from riverine or tidal flooding	● Levee failure during major flood will require costly emergency measures (emergency embankments; sand bags) and rebuilding of structure
	● Require periodic maintenance and inspections
	● Special seepage cutoffs or interior drainage structures (e.g., pressure conduits) will add to construction costs

Where seepage basins are used (Figure 3-24), runoff will be intercepted by a series of diversions, or the like, and passed to the basins. As illustrated, the recharge basin should consist of the actual basin, a sediment trap, a by-pass for excess runoff, and an emergency overflow. A considerable amount of recharge occurs through the sidewalls of the basin, and it is preferable that these be constructed of pervious material. Gabions are frequently used to make sidewalls. An alternative design for a seepage basin is shown in Figure 3-25. This is not designed for as intensive recharge as the previously discussed system, and is usually used where the aquifer is shallow.

Dense turf on the side slopes of these basins will prevent erosion and sloughing and will also allow a high infiltration rate. Prevention of scour by the inlet is an important consideration since it can significantly reduce

FIGURE 3-24

SEEPAGE BASIN; LARGE VOLUME, DEEP DEPTH TO GROUNDWATER

(SOURCE: Tourbier and Westmacott, 1974)
See Copyright Notice, Page 496

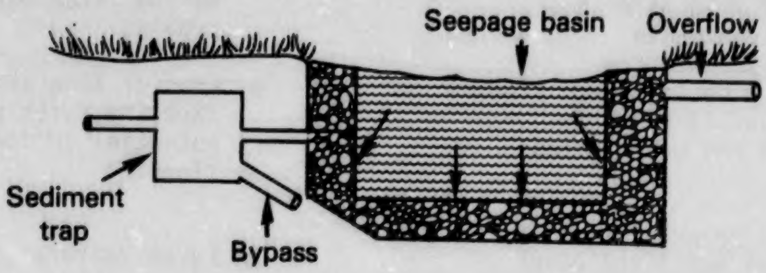
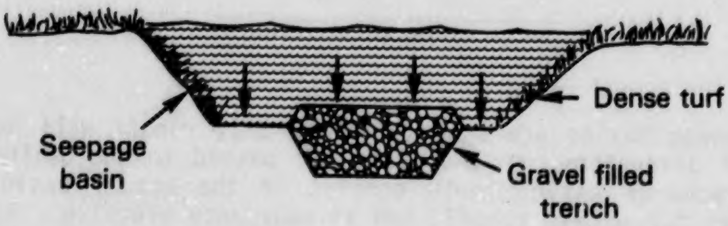

Seepage basin
Overflow
Sediment trap
Bypass

FIGURE 3-25

SEEPAGE BASIN; SHALLOW DEPTH TO GROUNDWATER

(Source: Tourbier and Westmacott, 1974)


Dense turf
Seepage basin
Gravel filled trench

maintenance requirements. This can be accomplished by a "hydraulic jump" or an impact stilling basin before water flows into the recharge basin. Percolation can be improved by construction of gravel-filled trenches along the basin floor, as shown in Figure 3-24 (Tourbier and Westmacott, 1974).

Use of seepage ditches (Figures 3-26 and 3-27) distributes water over a larger area than can be achieved with basins. They can be used for all soil where permeability exceeds about 0.9 in/day. Runoff is disposed of by a system of drains set in ditches of gravel. Depth and spacing of drains depends on soil permeability. A minimum depth of 48 inches is generally recommended, and ditches are rarely less than 10 feet apart. The ditches are backfilled with gravel, on which the distribution line is laid. Sediment is removed prior to discharging runoff into the seepage ditches by use of a sediment trap and distribution box. The efficiency of the seepage area can be increased by interconnecting two trenches by a continuous 12-inch gravel bed, as shown in Figure 3-27 (Tourbier and Westmacott, 1974).

FIGURE 3-26

SEEPAGE DITCH

(Source: Tourbier and Westmacott, 1974)

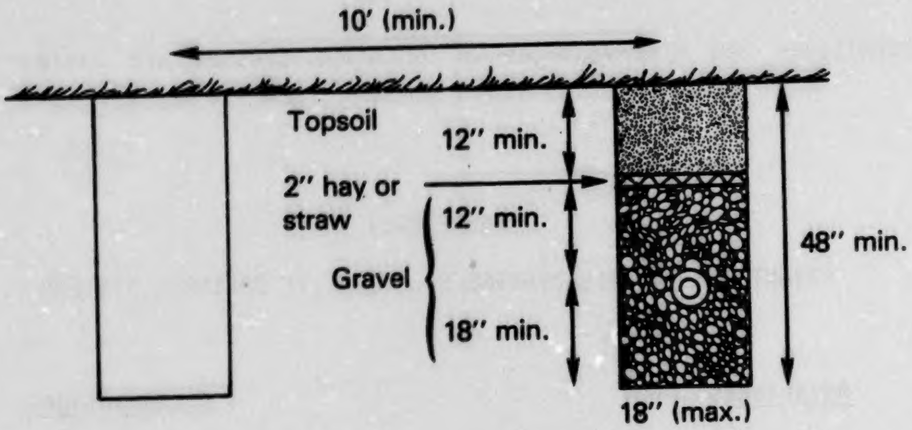
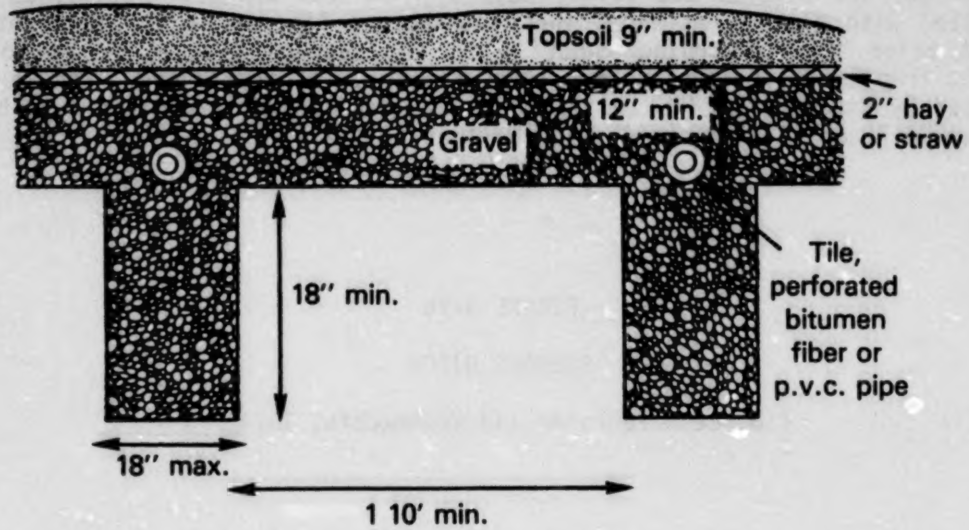


FIGURE 3-27

SEEPAGE DITCH WITH INCREASED SEEPAGE EFFICIENCY

(Source: Tourbier and Westmacott, 1974)


Topsoil 9" min.
12" min.
Gravel
2" hay or straw
Tile, perforated bitumen fiber or p.v.c. pipe
18" min.
18" max.
1 10' min.

3.4.6.3 Advantages and Disadvantages

Advantages and disadvantages of drainage systems are listed in Table 3-13.

TABLE 3-13

ADVANTAGES AND DISADVANTAGES OF GRAVITY DRAINAGE SYSTEMS

Advantages	Disadvantages
Cost effective means of intercepting runoff and allowing it to recharge	Seepage basins and ditches are susceptible to clogging
Systems can perform reliably if well maintained	Deep basins or trenches can be hazardous
	Not effective in poorly permeable soils

3.4.7 Sedimentation Basins/Ponds

3.4.7.1 General Description and Application

Sedimentation basins are used to control suspended solids entrained in surface flows. A sedimentation basin is constructed by placing an earthen dam across a waterway or natural depression, or by excavation, or by a combination of both. The purpose of installing a sedimentation basin is to impede surface runoff carrying solids, thus allowing sufficient time for the particulate matter to settle. Sedimentation basins are usually the final step in control of diverted surface runoff, prior to discharge into a receiving water body. They are an essential part of any good surface flow control system and should be included in the design of remedial actions at waste disposal sites.

3.4.7.2 Design and Construction Considerations

The removal of suspended solids from waterways is based on the concept of gravitational settling of the suspended material.

The size of a sedimentation basin is determined from characteristics of flow such as the particle size distribution for suspended solids, the inflow concentration, and the volumetric flow rate. To calculate the area of the sedimentation basin pond required for effective removal of suspended solids, the following data on the flow characteristics are needed:

- The inflow concentration of suspended solids.
- The desired effluent concentration of suspended solids. The desired effluent concentration is usually regulated by local and/or Federal government authorities. For example, for coal mines, the proposed EPA "Effluent Guidelines and Standard" limits are as follows: (1) total suspended solids concentration maximum for any one day is to 70 mg/l, and (2) average daily values for 30 consecutive days shall not exceed 35 mg/l.
- The particle size distribution for suspended solids.
- The water flow rate (Q) to the pond. For a pond receiving direct runoff, the runoff volume over a certain period of time must be determined. As an example, EPA has chosen the 10-year, 24-hour precipitation event as a design criteria for the overflow rate determination.

The steps in calculating the required area of the sedimentation basin are as follows:

(1) Calculate the removal efficiency of the pond by using the following formula:

$$R(\%\ \text{solids removed}) = 1 - \frac{10^6/C_1 - 1}{10^6/C_2 - 1} \times 100$$

where C_1 = solids concentration in influent (mg/l)
C_2 = solids concentration in effluent (mg/l)

(2) Determine the smallest size of particle that must be removed to achieve the required removal efficiency. The size of the removed particle can be graphically obtained from a particle size distribution for the suspended solid in the influent to the pond. Figure 3-28 shows a typical "particle size distribution" graph.

(3) Once the particle size is chosen, the settling velocity associated with the selected particle size can then be calculated using Stoke's law:

$$V_s = \frac{g}{18\mu}(S-1)D^2$$

where:
V_s = settling velocity (cm/sec)
g = gravitational acceleration (981 cm/sec^2)
μ = kinematic viscosity of the fluid (cm/sec^2)
S = the specific gravity of the particle
D = diameter of the particle (cm), assumed on sphere

(4) With the obtained water flow rate to the pond and the settling velocity, surface area of the pond can then be calculated as follows:

$$A = \frac{Q}{V_s}$$

where:
A = required area of the pond (m^2)
Q = the volumetric flow rate through the pond (overflow rate) (m^3/sec)
V_s = the critical settling velocity (m/sec)

(5) The final step is to multiply the required area of the pond by a safety factor of 1.2 to account for non-ideal settling:

$$A_{adjusted} = 1.2\ A_{required}$$

A typical installation of a sedimentation basin embankment is illustrated in Figure 3-29. As shown in this figure, the pond consists of a dike which retains the polluted water flow. For water drawdown purposes, a principal spillway is also needed.

FIGURE 3-28

PARTICLE SIZE DISTRIBUTION GRAPH

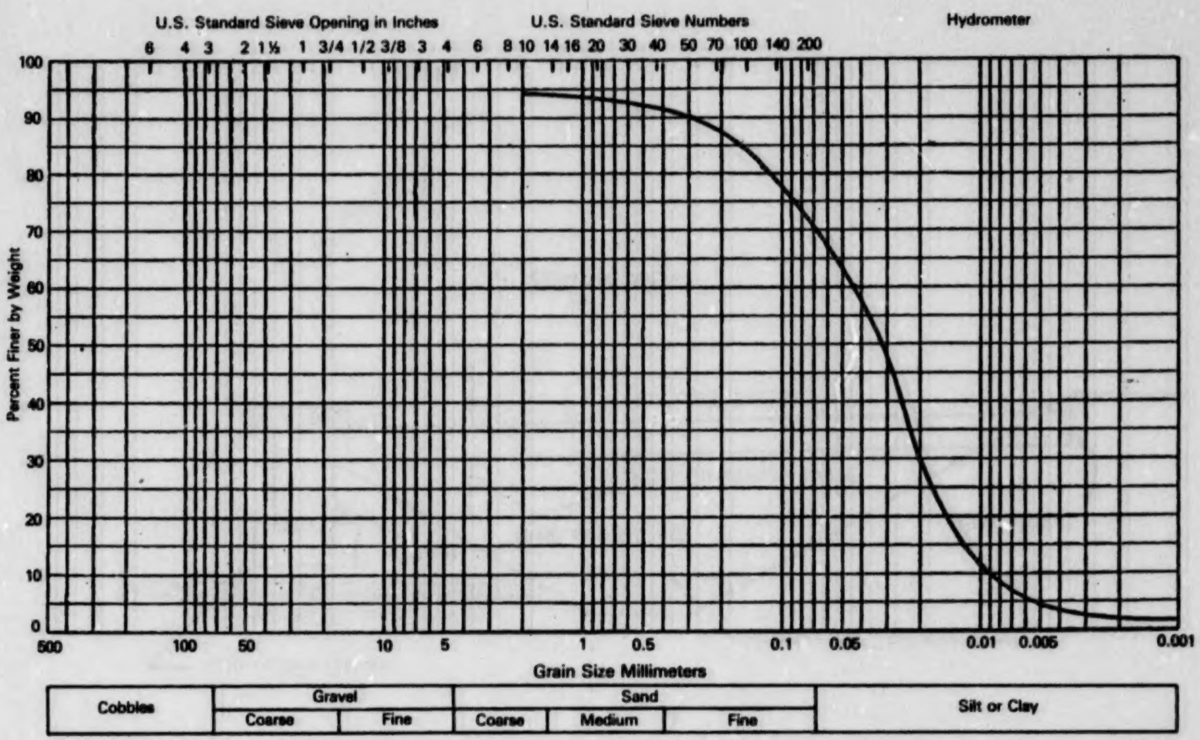
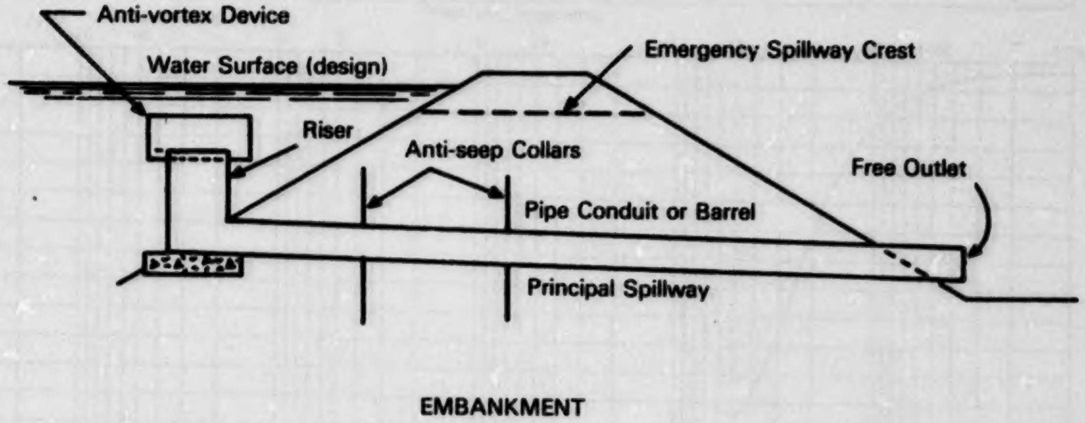


FIGURE 3-29

TYPICAL DESIGN OF A SEDIMENT BASIN EMBANKMENT


Anti-vortex Device
Water Surface (design)
Emergency Spillway Crest
Riser
Anti-seep Collars
Free Outlet
Pipe Conduit or Barrel
Principal Spillway
EMBANKMENT

The principal spillway consists of a vertical pipe or riser joined to a pipe that extends through the dike and outlets beyond the water impoundment. The riser is preferred to be topped by an antivortex device and trash rack. The riser should be watertight and, except for the dewatering opening at the top, it should not have any holes, leaks, or perforations. The riser base should be attached to a watertight connection and have sufficient weight to prevent flotation of the riser.

The water discharged from the sediment basin through the principal spillway should be conveyed in an erosion-free manner to an existing stable stream. Thus, at the discharge end of the spillway pipe, an impact basin, riprap, excavated plunge pools, and revetment should be constructed as protective measures against scour.

Emergency spillways are also suggested in the design of a sediment basin. They are provided to convey large flows safely past an earth embankment, and they are usually open channels excavated in earth, rock, or reinforcement concrete.

The efficiency of sedimentation ponds varies considerably as a function of the overflow rate. Sedimentation ponds perform poorly during periods of heavy rains and cannot be expected to remove the fine-grained suspended solids (Rogoshewski, 1978). If the sedimentation pond is expected to remove sediments that may have been contaminated by waste materials, consideration should be given to improving removal efficiencies by modifying basin or outlet design. One such modification to the outlet structure is shown in Figure 3-30. It is essentially a dual media sand filter surrounding the riser pipe. Other possible design modifications include the use of baffles, extra wide inflow or outflow weirs, energy dissipators, and siphon drawdown riser pipes (Rogoshewski, 1978). Alternatively, a two-pond system could be considered that should significantly increase removal efficiencies.

The quantity of material to be stored is also an important consideration in the construction of the sedimentation basin. The required storage capacity can be calculated by multiplying the total area disturbed by a constant sediment yield rate. Table 3-14 shows the storage requirements for some states. In some states, such as Maryland and West Virginia, there is a requirement to clean the sedimentation basin when the sediment accumulation reaches a specified limit imposed by the state. As an example, in West Virginia, the law requires cleaning of the sedimentation pond when the level of accumulation is 60 percent of the design capacity.

TABLE 3-14

DESIGN STORAGE CAPACITY REQUIREMENTS FOR SEDIMENTATION BASINS

State	Requirement
Maryland	0.5 inches/acre drained 0.2 inches/acre drained[1]
Kentucky	0.2 acre-ft/acre disturbed
West Virginia	0.125 acre-ft/acre disturbed 60 percent[2]
Pennsylvania	$V = (AIC) + (AIC/3)$ v = volume (ft^3) A = area drained I = rainfall/24 hours C = runoff constant

---

[1] To be cleaned when storage capacity drops below 0.2 inches/acre drained.

[2] To be cleaned when sediment accumulation approaches 60 percent design capacity.

(Source: EPA, 1976)

3.4.7.3 Advantages and Disadvantages of Sedimentation Basins

The advantages of the sedimention basin in the control of water flow contaminated with suspended solids can be listed as follows:

- Easy to design and install
- Requires low operational and maintenance effort
- Very effective in the removal of suspended solids

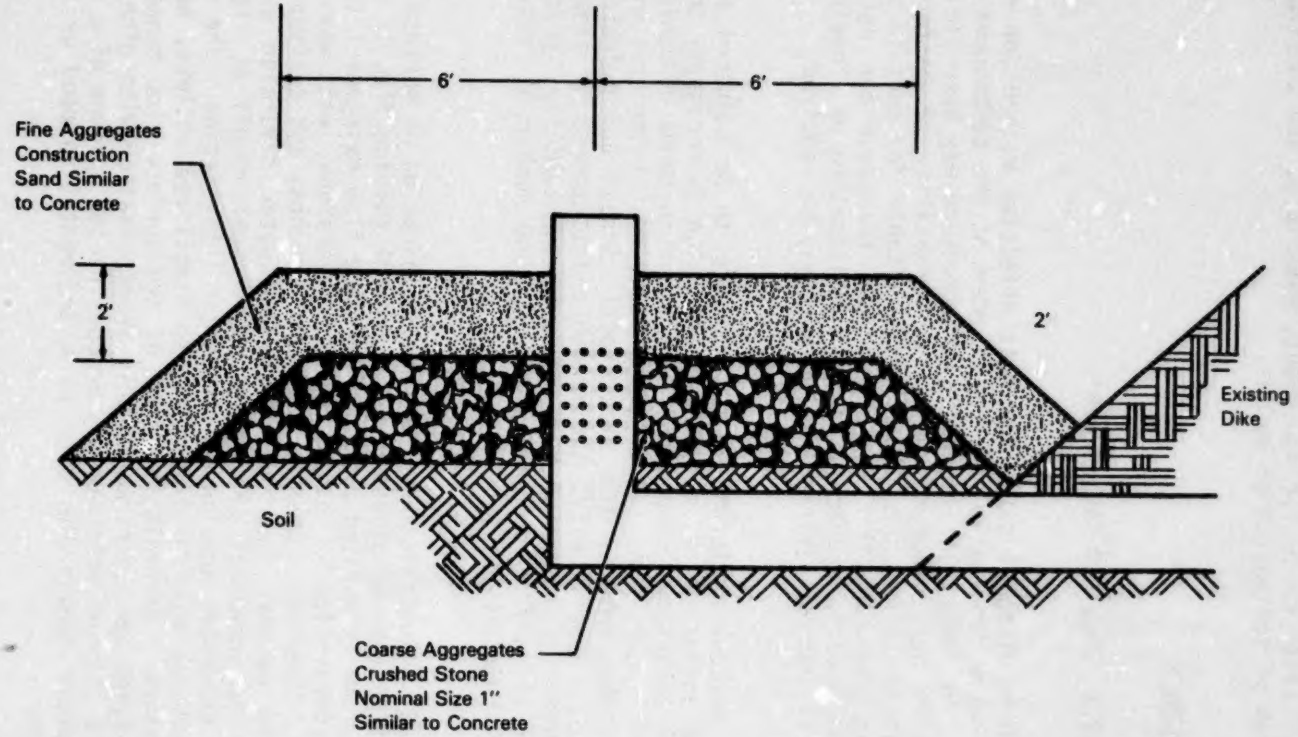
The major disadvantage of this method can be identified as follows:

- Other than the removal of suspended solids, it can not be used to remove other contaminants such as organic or inorganic chemicals
- Faulty design of the embankment or failure of the structure may result in damages to properties and life as well as interruption of the use of public utilities

FIGURE 3-30

MODIFICATION OF CONVENTIONAL SEDIMENTATION POND TO REDUCE SUSPENDED SOLIDS

(Source: Rogoshewski, 1978)


6'
6'
Fine Aggregates
Construction
Sand Similar
to Concrete
2'
2'
Existing
Dike
Soil
Coarse Aggregates
Crushed Stone
Nominal Size 1"
Similar to Concrete

3.4.7.4 Cost

Table 3-15 lists costs for equipment, materials, and construction needed for installation of sedimentation basins.

3.4.8 Summary

3.4.8.1 Combined Techniques

At any given disposal site, the most effective method for managing surface flow may be a combination of two or more of the techniques discussed in this chapter. An individual technique or structure may have limited application in terms of permanence of function, versatility of performance, topographic/hydrologic limitations (e.g., only suitable for certain size drainage area), and required capital. Where a single technique can not effectively control a site-specific problem (erosion, infiltration or flooding), an integrated system of several different techniques may be required.

Whenever several different structures are to be combined at a disposal site to manage surface flow, a site-specific plan is necessary to ensure that the individual techniques complement each other in terms of design, construction, and performance. The selection of individual techniques will depend on the size and topography of the site, local climate and hydrology, and soil characteristics. Specifically, the length and steepness of slopes, the frequency and intensity of rainfall, and soil permeability, erodibility, and fertility will all affect the choice of type and number of individual structures to be included at the site.

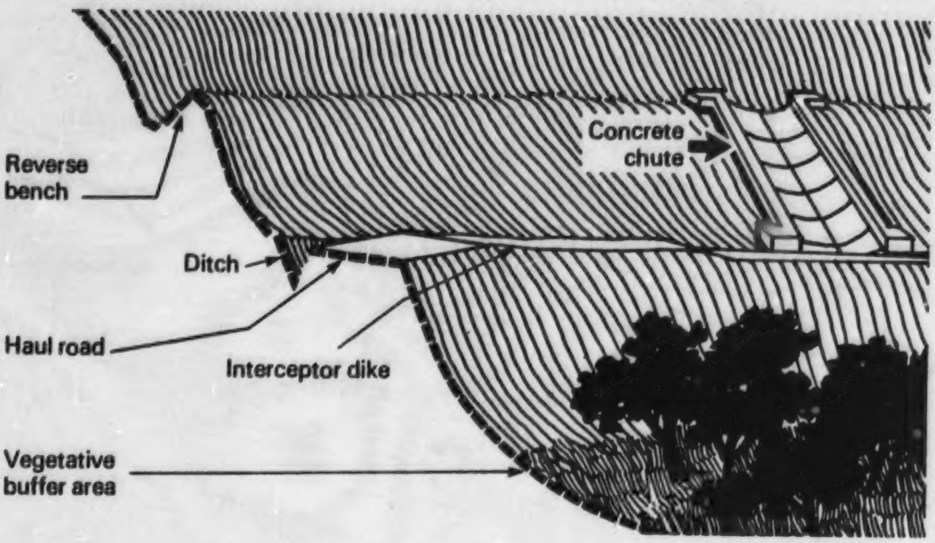
Figure 3-31 illustrates one possible combination of surface water interception and diversion techniques used to control surface flow in one area of a critical site. An integrated system of surface flow management techniques may include any combination of dikes, berms, diversions, waterways, bench terraces, chutes, downpipes, levees, drainage trenches, and sedimentation basins or traps. The general function of surface water diversion and collection structures is to intercept or detain and channel runoff at flow rates that cause neither excessive erosion nor excessive infiltration. The water carried by these structures must be discharged to stabilized outlets, holding ponds, or natural waters. Accordingly, some of the surface flow management structures are designed and constructed only in combination with other structures. For example, a grassed waterway may be constructed upslope of a disposal site to channel runoff around the site to a sedimentation pond or seepage basin locate offsite.

For very long or very steep slopes on critical sites, several diversions or several drainage benches may be constructed along the contour with the collected runoff being channeled to two grassed waterways draining downslope

FIGURE 3-31

INTEGRATED SYSTEM OF INTERCEPTION AND DIVERSION TECHNIQUES

(Source: EPA, 1976)



along either side of the site. The waterways may outlet to level spreaders stabilized with vegetation or riprap; the spreaders function to convert the concentrated flow of the waterways to non-erosive sheet flow (Figure 3-32).

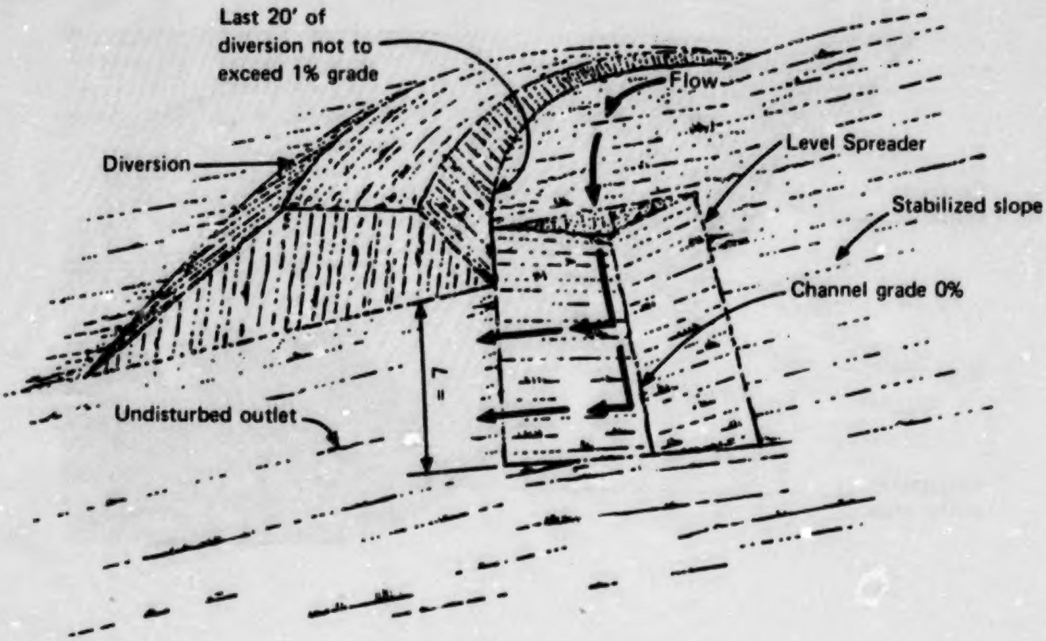
3.4.8.2 Costs

The costs of excavation, hauling, backfill, grading (spreading, compaction), vegetative stabilization, and maintenance are common to almost all surface flow control techniques. Other costs associated with surface water diversion/collection structures include those for ditch linings, riprap slope protection, soil testing, corrugated metal pipe, sheet piling, backflow valves, and sumps. Table 3-15 summarizes these unit costs, and the structure construction they apply to.

FIGURE 3-32

LEVEL SPREADER

(Source: EPA, 1976)


Last 20' of diversion not to exceed 1% grade
Flow
Diversion
Level Spreader
Stabilized slope
Channel grade 0%
L =
Undisturbed outlet

All cost estimates should be determined on a site-specific basis, considering the specific structures to be installed, all associated earthwork, and any special appurtenances that may be required. A general methodology for estimating costs for construction of surface water diversion/collection structures will contain the following elements:

- Determine source of required earth fill; on-site vs. off-site, and hauling distances
- Determine amount of fill required (yd^3)
- Determine type and quantity of other materials required (yd^3 of pipe, ft^2 of riprap, etc.)

- Determine costs of installation or placement of these materials using unit costs

- Determine costs of required stabilization for earthen structures (levee, berms, etc.) based on area in yd^2 to be stabilized; revegetation, riprap, or gravel stabilization

- Determine required maintenance or repair costs for a given time period based on reasonable assumptions; for example: assuming the diversion requires rebuilding (new fill and compaction) two times/year after major storms, then costs will be...

- Add all calculated costs for total estimated construction and maintenance expenditures

Costs are derived simply by multiplying unit costs (shown in Table 3-15) by required quantities of the material or service. These costs will give gross estimates only; they are to be used as general guides for the decision-maker in evaluating alternative remedial action strategies.

TABLE 3-15

UNIT COSTS ASSOCIATED WITH SURFACE WATER DIVERSION AND COLLECTION STRUCTURES

Description	Applicable structures[1]	Unit cost	Source of cost data
● Excavation, hauling, grading (spreading and compaction)	All	See Tables 3-2 and 3-4	
● Trench excavation;	D/B; D/D/W; BT, L; DT/B		
Loam, sand, and loose gravel			
1' -6' deep, ½:1 sides		\$.39-.67.yd^3	2
6'-10' deep		\$.39-52/yd^3	2
Compacted gravel and till			
1'-6' deep, ½:1 sides		\$.39-.69/yd^3	2
6'-10' deep		\$.30-.49/yd^3	2
● Building embankments; spreading, shaping, compacting; material delivered by scraper	All	\$.19-.38/yd^3	2
material delivered by back dump		\$.45-.63/yd^3	2
● Placement of ditch liner pipe; 1/3 section, 15" radius	D/D/W; drainage benches; C/D	\$9.6/ft	2
18" radius		\$14.7/ft	2
24" radius		\$18.4/ft	2
● Catch basin sump, 3'x4'x1.5'	L; DT/B	\$148 each	2
● Corrugated galvanized steel underdrain pipe, asphalt-coated, perforated;	DT/B		

--continued--

TABLE 3-15 (Continued)

Description	Applicable structures[1]	Unit cost	Source of cost data
12" diameter, 16 gage		$12/ft	2
18" diameter, 16 gage		$17/ft	2
● Corrugated galvanized metal pipe, with paved invert	C/C; SB		
18" diameter, 14 gage		$19.75/ft	2
36" diameter, 12 gage		$50.4/ft	2
48" diameter, 12 gage		$66.8/ft	2
● Steel sheet piling;	L (seepage control)		
15' deep, 22 psf		$8.15/ft^2	3
20' deep, 27 psf		$9.50/ft^2	3
25' deep, 38 psf		$12.20/ft^2	3
● Backflow preventer; gate values, automatic operation, flanged, 10" diameter	L (drainage control)	$8,900 each	3
● Floating baffles	SB	$20-50/ft	4
● Sump pumps; 6"-12" centrifugal pumps, operating 1 shift/day	L (backwater drainage)	$165-240/day	2
● Revegetation, mulching, maintenance	D/B; D/D/W;BT; L	See Table 3-8	
● Loose gravel, excavation, loading, hauling 5 miles, spread and compacting	All (slope protection; drainage)	$4-4.50/yd^3	2

--continued--

TABLE 3-15 (Continued)

Description	Applicable structures[1]	Unit cost	Source of cost data
● Stone riprap; dumped from trucks, machine-placed	All (slope protection; channel & outlet stabilization)	$\$16.65/\text{yd}^3$	3
● Soil testing;	All (preconstruction evaluation)		
liquid and plastic limits		\$35/test	3
hydrometer analysis; specific gravity		\$60/test	3
moisture content		\$ 8/test	3
permeability		\$50/test	3
Proctor compaction		\$40-45/test	3
shear tests, triaxial		\$195-350/test	3
direct shear		\$75-225/test	3
● Temporary sediment construction; drainage area,	SB		
1-25 acres		\$300-1600 each	5
50-75 acres		\$3000-5000	5
75-100 acres		\$5000-[illegible]00	5
100-125 acres		\$6400[illegible]000	5
● Sediment removal from basins	SB	$\$3\text{-}7/\text{yd}^3$	5
● Paved flume, installed	C/D; SB	$\$20\text{-}30/\text{yd}^2$	5
● Level spreader construction	D/B; D/D/W; BT; C/D	\$2.50-5.00/ linear foot	5

---

[1] Key: D/B, dikes and berms; D/D/W, ditches, diversions, and waterways; BT, bench terraces; C/D, chutes and downpipes; L, levees; DT/B, drainage trenches and basins; SB, sediment basins.

[2] McMahon and Pereira, 1979.

[3] Godfrey, 1979.

[4] Environetics, Inc., 1980.

[5] Virginia Soil and Water Conservation Commission, 1974.

REFERENCES

Brunner, D., and D. Keller. 1972. Sanitary landfill design and operation. U.S. Environmental Protection Agency. Report SW-65

Environetics, Inc., Bridgeview, IL. 1980..Personal communication with Carl Klikas.

Fields, T. Jr., and A. W. Lindsey. 1975. Landfill disposal of hazardous wastes: A review of literature and known approaches. U. S. Environmental Protection Agency report, EPA/530/SW-165. Office of Solid Waste Management Programs, Washington, D.C.

Fisher, G., DuPont de Nemours & Co., Inc. Wilmington, DE. February 1980. Personal communication with G. Hunt.

Gilman, E., F. Flower, I. Leone, and J. Arthur. 1979. Vegetation growth in landfill environs in municipal solid waste. In: Land Disposal Proceedings of the Fifth Annual Research Symposium. Wanielista, M. and J. Taylor (eds.). Cincinnati, Ohio: Municipal Environmental Research Laboratory, Office of Research and Development. EPA-600/9-79-023a.

Godfrey, R. (ed.). 1979. Building construction cost data, 1980. Kingston, MA: Robert Snow Means Company, Inc.

Haseley, M., Haseley Trucking Company, New York, NY. February 1980. Personal communication with D. Twedell.

JRB Associates, Inc., and Emcon Associates. 1980. Assessment of alternatives for upgrading Navy solid waste disposal sites. Draft Final Report. McLean, VA.

Linsley, R., and J. Franzini. 1979. Water resources engineering, 3d ed. New York: McGraw-Hill Book Company.

Lutton, R. 1978. Selection of cover for solid waste in land disposal of hazardous wastes. Proceedings of the Fourth Annual Research Symposium. D. Shultz (ed.). Cincinnati, Ohio: Municipal Environmental Research Laboratory, ORD. EPA-600/9-78-016.

Lutton, R., G. Regan, and L. Jones. 1979. Design and construction of covers for solid waste landfills. Cincinnati, Ohio: Municipal Environmental Research Laboratory, ORD. EPA-600/2-79-165.

McMahon, L., and P. Pereira. 1979. 1980 Dodge guide to public works and heavy construction costs. New York: McGraw-Hill Information Systems.

Molz, F., S. Van Fleet, and V. Browning. 1974. Transpiration drying of sanitary landfills. Groundwater 12(6): 394-398.

Nawrocki M. 1976, Removal and separation of spilled hazardous materials from impoundment bottoms. Cincinnati, Ohio. EPA-600/2-76-245.

Rogoshewski, P. 1978. Construction source sediment control. Washington, D.C.: USEPA, Office of Research and Development.

Tolman, A., A. Ballestero, W. Beck, and G. Emrich. 1978. Guidance manual for minimizing pollution from waste disposal sites. Cincinnati, OH: U.S. Environmental Protection Agency. Cincinnati, OH. EPA-600/2-78-142.

Tourbier, J., and R. Westmacott. 1974. Water resources protection measures in land development--a handbook. Newark, Delaware: Water Resources Center, University of Delaware.

U.S. Environmental Protection Agency. 1976. Erosion and sediment control, surface mining in the eastern U.S., vol. 1: Planning; vol. 2: Design. Washington, D.C.: EPA Technology Transfer. EPA-625/3-76-006.

U.S. Environmental Protection Agency. 1979. Hazardous waste: proposed guidelines and regulations and proposal on identification and listing. Federal Register 43(243): 59011. December 19, 1979.

Universal Linings, Inc., Philadelphia, PA. 1980. Personal communication between D. Small and P. Rogoshewski.

Virginia Soil and Water Conservation Commission. 1974. Virginia erosion and sediment control handbook: standards, criteria and guidelines. Richmond, Virginia: Virginia SWCC.

Wilson, D.G. (ed.). 1977. Handbook of solid waste management. New York: Van Nostrand Reinhold Co.

4.0 GROUNDWATER CONTROLS

Groundwater that has been contaminated by an uncontrolled waste disposal site can be dealt with in a number of ways. Impermeable barriers constructed of bentonite slurry, cement or chemical grouts, or sheet piling can be installed vertically to (1) prevent groundwater from migrating away from the site; or (2) divert groundwater so that contact with waste materials is prevented. Another potential method of dealing with contaminated groundwater is to allow it to flow through permeable treatment beds (limestone and/or activated carbon) in which the contaminants would be removed as the groundwater flowed through the bed. The above two treatment methods can be considered passive groundwater control.

The pumping of groundwater with subsequent surface treatment is considered an active remedial measure. Pumping of groundwater can be specifically designed to lower the groundwater table in the area of a disposal site or it can be designed to contain a contaminated groundwater plume.

The above-mentioned methods for control of contaminated groundwater are discussed in this chapter.

4.1 IMPERMEABLE BARRIERS

Impermeable barriers can be used to divert groundwater flow away from a waste disposal site or to contain contaminated groundwater emanating from a waste site. Various methods and materials that can be used to construct impermeable groundwater barriers are discussed in the following sections.

Before selection of an impermeable barrier to control groundwater flow, it should be recognized that impeded groundwater flow may cause an increase in upgradient hydraulic head, with consequent associated effects on rates of vertical movement of the water. The probable effects of a locally heightened water table should be carefully considered before deciding to apply this method of control.

4.1.1 Slurry Walls

4.1.1.1 General Description

Slurry trenching is a method of constructing a subsurface barrier or slurry wall to reduce or redirect the flow of groundwater. This technique was pioneered in the United States in the mid-1940's using technology developed by the oil industry (Boyes, 1975). Today, this practice covers a range of construction techniques from the simple to the quite complex, and though it is becoming more common, is still performed by only a few specialty contractors. In recent years, engineers and contractors have become aware of the low cost and nearly universal success of slurry trench cut-offs, and this technique has largely replaced other methods such as grout curtain cut-offs and sheet piling cut-offs (D'Appolonia, 1979).

In general, slurry trenching involves excavating a trench through or under a slurry of bentonite clay and water, and then backfilling this trench with the original soil with or without slurry mixed in. Most commonly, the trench is excavated down to, and often into, an impervious layer in order to shut off groundwater flow. This may not be the case when only a lowering of the water table is required. The width of the trench can vary, but is typically from 2 to 5 feet (D'Appolonia, 1979). Depending on the depth of the trench, light or heavy equipment is used for excavation.

The slurry used in this practice is essentially a 4 to 7 percent by weight suspension of bentonite in water (Boyes, 1975). Bentonite is a clay of the montmorillonite group of 2:1 expanding lattice clays. The silica and alumina mineral lattice or layers of these clays can expand and contract depending on the amount of water and the interlayer cations present. In bentonite, the interlayer cations are primarily sodium, whose large hydration radius causes maximum swelling of the clay layers (Baver et al., 1972). Bentonite is mined in the western United States, principally in Wyoming, and is often called Wyoming bentonite.

Excavation of a trench under a bentonite slurry causes two things to happen. First, the slurry acts as shoring, supporting the trench walls to prevent cave-ins and slumping during further excavation. Secondly, and most importantly, the weight of the slurry forces bentonite into the soil matrix on the trench walls and bottom. As more and more bentonite is forced into the soil, a filter cake is formed, the thickness of which depends on the permeability of the soil and the weight of slurry. In essence, the trench becomes completely lined with a layer of soil and bentonite of extremely low permeability.

When the trench has been excavated to the desired depth, backfilling is begun. Often one section of trench is backfilled while a new section is being excavated. Backfilling is sometimes done with only the excavated soil material, but most often with a soil and bentonite mixture. In some cases, additional soil or Portland cement is added to achieve the desired result. This is discussed in more detail below. The end result of this procedure is a wall that is impervious or of negligible permeability.

4.1.1.2 Applications

Slurry walls were first used to effect groundwater cut-off in conjunction with large dam projects. In recent years, they have found use as both groundwater and leachate barriers around hazardous waste disposal sites. Placement of the wall depends on the direction and gradient of groundwater flow as well as location of the wastes.

When placed on the upgradient side of a waste site, a slurry wall will force the groundwater to flow around the wastes. In some instances, it may be unnecessary to sink the wall down to an impervious stratum. A wall sunk far enough into the water table upgradient from the wastes can reduce the head of the groundwater flow, causing it to flow at greater depth beneath the wastes. In either case, groundwater flow through the wastes is virtually eliminated and the production of hazardous leachate is greatly reduced.

In certain settings, such as in tidal areas or along major rivers, the direction of groundwater flow can periodically reverse. In this case, or when an extreme hazard (posed by pollutants such as dioxin) requires total isolation, a slurry wall can be installed to completely surround the wastes. The slurry wall must come in contact not only with groundwater, but with chemical-laden leachate as well. Tests have been conducted to determine the ability of bentonite slurry walls to withstand the effects of certain pollutants, and the results are encouraging. As can be seen in Table 4-1, of the chemicals tested, only alcohols were found to completely destroy the slurry wall. To determine the probable effectiveness of a slurry wall for a particular site, however, tests should be conducted using the acutal leachate from the site.

In cases where the permeability of the bentonite is found to decrease in the presence of leachate, the admixture of polymer compounds to the slurry may prevent the breakdown of the retaining properties of the slurry wall.

4.1.1.3 Design and Construction Considerations

From the above factors, it should be evident that slurry trenching must be preceded by thorough hydrogeologic and geotechnical investigations. A good hydrogeologic study will tell the designers the depth, rate, and direction of groundwater flow, and the chemical characteristics of the water. A geotechnical investigation will provide information on soil characteristics such as

TABLE 4-1

PERMEABILITY INCREASE DUE TO LEACHING WITH VARIOUS POLLUTANTS

Pollutant	Filter cake	SB backfill (silty or clayey sand) 30 to 40% fines
CA^{++} or Mg^{++} @ 1,000 PPM	N	N
CA^{++} or Mg^{++} @ 10,000 PPM	M	M
NH_4NO_3 @ 10,000 PPM	M	M
HCL (1%)	N	N
H_2SO_4 (1%)	M	N
HCL (5%)	M/H[1]	M/H[1]
NaOH (1%)	M	M
CaOH (1%)	M	M
NaOH (5%)	M	M/H[1]
Sea Water	N/M	N/M
Brine (SG=1.2)	M	M
Acid Mine Drainage $FeSO_4$ (PH-3)	N	N
Lignin (in Ca^{++} solution)	N	N
Alcohol	H (failure)	M/H

N - No significant effect; permeability increase by about a factor of 2 or less at steady state.
M - Moderate effect; permeability increase by factor of 2 to 5 at steady state.
H - Permeability increase by factor of 5 to 10.
[1] Significant dissolution likely.
(Source: D'Appolonia, 1980)

permeability, amount of stratification, and depth to bedrock or an impervious layer. In addition, it will tell the nature and condition of the bedrock. When the slurry wall is intended to provide total water cut-off, rather than just to lower the water table, particular attention must be paid to the soil/rock interface. It may be necessary to excavate several feet into the bedrock to ensure the integrity of the wall. Another method of ensuring this integrity is to install the slurry wall as outlined above and to drill down through it into rock, and then to inject a grout solution to seal all voids. Although this is very effective, it is also quite costly. (See Section 4.1.2).

The type of equipment used to excavate a slurry trench depends primarily on the depth. Hydraulic backhoes can be used to excavate to around 55 feet. Beyond that depth, a clam-shell shovel must be used. If it is necessary to install the slurry wall into hard bedrock, drilling or blasting may have to be used to excavate the rock. Special blasting techniques would be required to maintain the integrity of the bedrock.

Backfilling of a trench is often accomplished with the equipment used to excavate the trench. A bulldozer is used to mix the soil with the slurry alongside the trench as well as to backfill the upper portion of the trench. Care must be taken to ensure that no pockets of slurry are trapped during the backfilling, as these can greatly reduce the wall's effectiveness and permanence.

For maximum permeability reduction, the soil/bentonite mixture used for backfilling should contain 20 to 25 percent fines (soil particles that will pass a 200-mesh sieve). To ensure long-term permeability reduction, as much as 40 to 45 percent fines may be required. In the event the on-site soils are too coarse, imported fines or additional bentonite must be added (Shallard, 1980).

4.1.1.4 Advantages and Disadvantages

The process outlined above includes a number of variables that can affect the long-term effectiveness of a slurry wall. The extent to which these variables, such as groundwater, soil, and rock characteristics, can influence the integrity of a wall, can usually be determined by a variety of preconstruction tests. From the results of these field and laboratory tests, more site deficiencies can be identifed and corrected prior to construction. A properly designed and installed slurry wall can be expected to provide effective groundwater control for many decades with little or no maintenance.

4.1.1.5 Costs

Slurry wall construction is a relatively high-technology technique, performed by only a few specialty firms. It is by no means inexpensive. The following example provides an idea of the costs involved:

> A slurry wall, 1,000 feet long and 3 feet wide, is to be placed down to bedrock along the upgradient side of a hazardous waste disposal site. Along this line, depth to hard bedrock averages 40 feet. In all, 4,440 cubic yards of slurry wall must be installed. The costs associated with construction of this wall are listed in Table 4-2.

TABLE 4-2

COSTS ASSOCIATED WITH SLURRY WALL CONSTRUCTION[1]

Activity	Unit costs (where applicable)	Total costs
Testing--geotechnical, hydrologic, and lab filter cake permeability	N.A.	$20,000-$80,000
Equipment mobilization--hydraulic backhoe, bulldozer, slurry mixer, etc.	N.A.	$20,000 - $80,000
Slurry trenching, excavation, mixing, and backfilling	$45 - $70 per cubic yard	Approximately $200,000 - $310,000
Overall	N.A.	$240,000 - $470,000

[1]Cost from a variety of industry sources (1980).

Note: These figures are for a 10-acre waste site located 150 miles from the trenching contractor. The upgradient portion of the site is given a thorough investigation; the downgradient portion, a cursory investigation; and the portion containing the wastes, no investigation.

4.1.2 Grout Curtains

4.1.2.1 General Description

Another method of groundwater control is the installation of a grout curtain. Grouting is, in general, the pressure injection of one of a variety of special fluids into a rock or soil body to seal and strengthen it. Once in place, these fluids set or gel into the rock or soil voids, greatly reducing the permeability of and imparting increased mechanical strength to the grouted mass. When carried out in the proper pattern and sequence, this process can result in a curtain or wall that can be a very effective groundwater barrier.

Because a grout curtain can be three times as costly as a slurry wall, it is rarely used when groundwater has to be controlled in soil or loose overburden. The major use of curtain grouting is to seal voids in porous or fractured rock where other methods of groundwater control are impractical.

The pressure injection of grout is as much an art as a science. The number of U.S. firms engaging in this practice is quite limited. The injection process itself involves drilling holes to the desired depth and injecting grout by the use of special equipment. In curtain grouting, a line of holes is drilled in single, double, or sometimes triple staggered rows (depending on site characteristics) and grouting is accomplished in descending stages with increasing pressure (Bowen, 1975). The spacing of the injection holes is also site-specific and is determined by the penetration radius of the grout out from the holes. Ideally, the grout injected in adjacent holes should touch between them (Figure 4-1). If this process is done properly, a continuous, impervious barrier (curtain) will be formed.

4.1.2.2 Applications

In general, grouts can be divided into two main categories--suspension grouts and chemical grouts. Suspension grouts, as the names implies, contain finely divided particulate matter suspended in water. Chemical grouts, on the other hand, are true Newtonian fluids. Most of the grouting in the United States is done with suspension grouts, while about half of the grouting in Europe is done with chemicals (Kirk-Othmer, 1979). The principal grouts in use today are briefly described below.

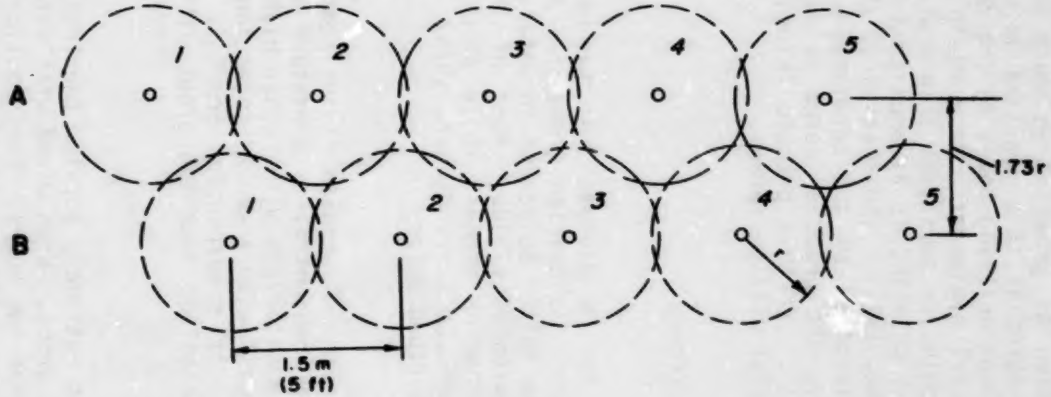
Suspension grouts are non-Newtonian fluids composed, for the most part, of either Portland cement, bentonite, or a mixture of the two. Their primary use is in sealing voids in materials with rather high permeabilities, and they are often used as "pregrouts" with a second injection of a chemical grout used to seal the fine voids. If a suspension grout is injected into too fine a medium, filtration of the solids from the grout will occur, thus eliminating its effectiveness.

Portland cement is an extremely popular construction material, and one of the first to see use as a grout. When mixed with water, it will set up into a crystal lattice in less than two hours. For grouting, a water-cement ratio of 0.6 or less is more effective (Bowen, 1975). The smallest voids that can be effectively grouted are no smaller than three times the cement grain size. For this, it is clear that a more finely ground cement makes a more watertight grout (Bowen, 1975). Portland cement is often used with a variety of additives that modify its behavior. Among these are clays, sands, fly-ash, and chemical grouts.

FIGURE 4-1

TYPICAL TWO-ROW GRID PATTERN FOR GROUT CURTAIN

(Source: EPA, 1978)


A
B
1
2
3
4
5
1
2
3
4
5
r
1.73r
1.5 m
(5 ft)

Of the clay minerals used for grouting, bentonite is by far the most common. Other locally available clays, especially those of marine or river origin, may be used but must be extensively tested and often chemically modified (Bowen, 1975). Bentonite, however, because of its extremely small particle size (one micron or less), is the most injectable, and thus the best suited for grouting (Tallard and Caron, 1977). Bentonite grouts can be injected into materials with lower permeabilities than can other suspension grouts. Medium- to fine-textured sands, with permeabilities of around 10^{-3} -10^{-4} cm/second, can be sealed with a bentonite grout (AFTES, 1975). Dry bentonite is mixed with water on-site at a rate of 5 to 25 percent by dry weight (Bowen, 1975; Tallard and Caron, 1977). In these ratios, bentonite will absorb large amounts of water and, with time, form a gel. This gel, although it imparts little if any structural strength, is an extremely effective water barrier.

Chemical grouts are a more recent development than suspension grouts. With the exception of one, silicate grout, they have been developed since World War II. Chemical grouts are true Newtonian fluids and can, depending on their nature and concentration, have low to very high viscosities. Some have viscosities approaching that of water and so can be injected virtually anywhere water can penetrate. Because of this, they can be used to waterproof very fine rock and soil voids. They are often used in conjunction with Portland cement or bentonite, to seal both large and small voids. This versatility can offset the high cost and complicated technology of injecting chemical grouts.

The oldest and still the most commonly used chemical grouts are silicate-based. Today, they account for over 75 percent of the grouting performed for waterproofing (Tallard and Caron, 1977). Silicate grouts are composed of a sodium silicate base, a reactant, an accelerator, and water. The reactant is typically an amide, an acid, or some polyvalent cation. A salt, such as calcium chloride or sodium aluminate, is used to accelerate the set or gel of the grout. The concentration of sodium silicate in the grout varies between 20 and 60 percent, and for waterproofing should be less than 30 percent (Tallard and Caron, 1977). As with many other grouts, the lower the concentration, the lower the viscosity will be. The concentration of the grout used will depend on the material into which it is to be injected, with the lower limit of injectability being soil with greater than 20 percent sand (Bowen, 1975).

Lignochrome grouts consist of lignosulfonate or lignosulfite reacted with a hexavalent chromium compound. Lignosulfonates and lignosulfites are byproducts of the wood pulp processing industry, and depending on the extraction process used, can be salts of calcium, ammonium, or sodium. For grout use, sodium lignosulfonates are too unstable, while calcium lignosulfonates provide the best waterproofing and the most stability (Tallard and Caron, 1977). Lignochrome grouts set up in an acid medium, where the highly toxic hexavalent chromium is reduced to the less toxic trivalent form. The set time of the grout is controlled by varying the concentrations of both the chromate and the accelerator (usually a metallic salt). Set times of from 10 minutes to 10 hours are possible (Tallard and Caron, 1977). Although grout viscosities low enough to be injected into fairly fine soils (10^{-3} cm/sec minimum) are obtainable with lignochromes, the viscosity increases gradually until set is achieved (AFTES, 1975). With most other grouts, the viscosity remains fairly constant until they set abruptly.

The use of acrylamide grouts was pioneered during World War II by the American Cyanamid Company. Their grout, AM-9, was the predominant acrylamid grout until production ceased in 1978. Since that time, a Japanese grout of slightly different formulation has been offered in this country as a replacement for AM-9. In general, acrylamide grouts are composed of acrylamide monomer; a cross-linking agent, methylene-bis-acrylamide; an accelerator, usually ammonium persulfate; and a catalyst (Bowen, 1975). American Cyanamid's grout used dimethylamino proprionitrile as a catalyst, and toxicity problems associated with it may well have led to its withdrawal from the market. The new acrylamide grouts use as a catalyst much less toxic triethanolamine (Avanti International, 1979). Still, because of the toxicity of the acrylamide monomer, care must be taken in mixing and injecting these grouts. Because of their low viscosities and rapid and controlled set times, these are among the most versatile grouts. They can be effectively injected into silty soils with permeabilities as low as 10^{-7} cm/sec (AFTES, 1975).

Phenoplasts are chemical grouts formed by the polycondensation of a phenol and an aldehyde (Tallard and Caron, 1977). By far the most commonly used phenoplast grout is resorcinol-formaldehyde. The reaction of these two compounds to form the set grout is achieved by an increase in the pH, with minimum set time obtained at a pH of 9.3 (Tallard and Caron, 1977). Set time can also be controlled by varying the dilution of the grout, with more dilute solutions taking longer to set (Tallard and Caron, 1977). Viscosities similar to those of acrylamide are obtainable with resorcinol-formaldehyde; hence the two types of grout have similar applications.

Aminoplasts are a class of grout that has seen little development and application. The major grout in this class is urea-formaldehyde. It forms a resin by a condensation reaction caused by heat in an acid medium. Because of the requirement for heat and a low pH, and their high viscosity, urea-formaldehyde grouts have been used very little (Tallard and Caron, 1977).

Grout curtains, because of their relatively high cost, are not the method of choice for groundwater control where a less expensive method, such as a slurry wall, is practical. Grouts are, however, the most practical and efficient method for sealing fissures, solution channels, and other voids in rock. As noted in Section 4.1.1, grouts can be very effective in insuring a watertight seal where a slurry wall is tied into bedrock or some other impermeable layer. Where rock voids allow the passage of large volumes of water, a grout can be formulated to set with sufficient speed to quickly shut off the flow. In theory, it is possible to place a grout curtain upgradient or downgradient from or beneath a hazardous waste site.

As with slurry walls, placing a grout curtain up the groundwater gradient from a waste site can redirect the flow so that groundwater no longer flows through the wastes that are creating hazardous leachate. Given a normal range of groundwater chemistry, most grouts could be expected to function well in this capacity.

Placement of a grout curtain downgradient from or beneath a hazardous waste site is quite another matter. Problems could be expected when attempting to grout in the presence of leachate or extreme groundwater chemistry. In many instances it would be difficult or impossible to control the set time, and consequently, to emplace a curtain of reliable integrity. Little information was found in the literature on the resistance of grouts to chemical attack. Should a case arise where grout must contact leachate or groundwater of extreme chemical characteristics, extensive tests would be have to be conducted. Additional problems occur in attempting to grout a horizontal curtain or layer beneath a waste site. In order to inject grout in such a case, injection holes must be drilled either directionally from the site perimeter, or down through the wastes. The first situation would be very expensive and the second could be very dangerous. In either case, it would be very difficult to place an effective barrier and virtually impossible to monitor its effectiveness.

4.1.2.3 Design and Construction Considerations

Pressure grouting is a high technology endeavor. As with slurry trenching, extensive geotechnical and hydrologic testing must precede the placement of a grout curtain. Boring, pumping, and laboratory tests will determine whether or not a site is groutable and will provide the necessary groundwater, rock, and soil information to allow for the choice of the best-suited grout or grouts. They will further provide the designer with the information needed to plan the pattern and procedure for injection. Other tests may be needed to evaluate leachate resistance, effectiveness of the grouting, or other factors.

The equipment used in pressure grouting is for the most part sophisticated special machinery. Much of this equipment is patented and/or proprietary. This machinery includes pumps and specialty drills for the boring of injection holes. Often, the pumps are connected to a manifold to allow grouting of several holes at once. In nearly all cases, the pumps are equipped with gages and meters to monitor grout pressures and volumes. Since only a few contractors perform this type of work, it is likely that equipment mobilization fees associated with grout curtain installation will be high.

4.1.2.4 Advantages and Disadvantages

When it becomes necessary to stop or reroute groundwater flowing in porous rock, pressure injection of grout may well be the only means. Grouts can be formulated to set within a few seconds, so that even rapidly flowing water can be shut off. Grout also can be used to control groundwater flow in soils, but in most cases, a more cost-effective method is available.

The drawbacks to grout usage stem from the fact that grouting is conducted by a limited number of firms in the United States and involves special techniques and equipment. In most cases, a substantial equipment mobilization fee must be paid. Equally important is the cost of pregrouting testing that must be performed to insure effective grouting. A final consideration is the cost of the grout itself. Approximate costs of the grouts discussed above are found in Table 4-3.

TABLE 4-3

APPROXIMATE COST OF GROUT[1]

Grout type	Approximate cost $/gallon of solution
Portland cement	0.95
Bentonite	1.25
Silicate - 20%±	1.75
- 30%	2.10
- 40%	2.75
Lignochrome	1.55
Acrylamide	6.65
Urea formaldehyde	5.70

[1] Costs taken from various industry sources (1980).

4.1.2.5 Costs

Some of the costs involved with grout curtain installation are illustrated by the following example: (Figure 4-2).

A 5-acre hazardous waste site is having leachate generation problems, and it is determined that a grout curtain should be placed upgradient from the wastes. The area chosen for the curtain is very shallow, hard, fractured rock; therefore, other control methods are not feasible (see Section 4.1.3). Site investigations show that an 800-foot-long wall, 20 feet deep, will solve the problem. The rock is found to have a permeability of 1×10^{-3} cm/sec, and a porosity of 30 percent. It is determined that injection holes will be placed 3 feet apart, and that a double row of holes is needed. The contractor feels a 20-percent silicate grout will effectively stop the groundwater flow. The porosity measurements show that for every cubic yard of curtain, 0.3 cubic yards of grout will be needed. The costs associated with installing this curtain are given in Table 4-4.

TABLE 4-4

COSTS ASSOCIATED WITH SILICATE GROUT CURTAIN PLACEMENT[1]

Activity	Unit cost (where applicable)	Total cost
Testing--geotechnical, hydrologic, site groutability	N.A.	\$10,000-\$40,000
Equipment mobilization--drills, mixers, pumps, manifolds, etc.	N.A.	\$20,000 - \$80,000
Grout in place	\$145/yd^3	\$154,640 (1066.5/yd^3)
Overall	N.A.	\$185,640 - \$247,640

---

[1] Based on parameters cited in example on page 128.

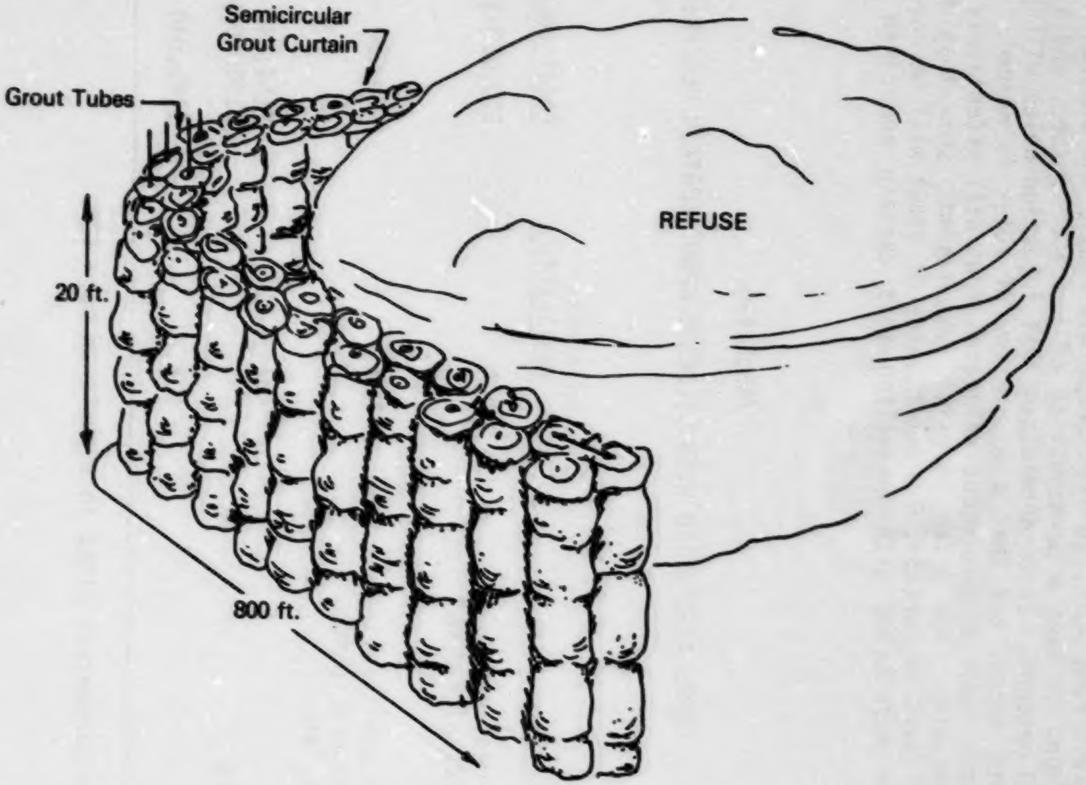
4.1.3 Sheet Piling Cut-off Walls

In addition to slurry wall and grout curtain cut-offs, sheet piling can be used to form a continuous groundwater barrier. Sheets piles can be made of

FIGURE 4-2

SEMICIRCULAR GROUT CURTAIN AROUND UPGRADIENT END OF LANDFILL

(Source: EPA, 1978)


Semicircular
Grout Curtain
Grout Tubes
REFUSE
20 ft.
800 ft.

wood, precast concrete, or steel. Wood is an ineffective water barrier, however, and concrete is used primarily where great strength is required. Steel is the most effective in terms of groundwater cut-off and cost, and so is discussed here.

4.1.3.1 General Description

The construction of a steel sheet piling cut-off wall involves driving interlocking piles into the ground with a pneumatic or steamdriven pile driver. In some cases, the piles are pushed into pre-dug trenches. Lengths of the piles are commonly from 4 to 40 feet, although longer lengths are available by special order (ARMCO, 1980). Nearly every manufacturer of steel sheet piling offers its own shape of piling and often its own form of interlock. Some of these are shown in Figure 4-3. Widths of sheet pile range from 15 to 20 inches (Ueguhardt, 1962).

4.1.3.2 Applications

In instances where wastes are deposited in contact with a permanent or seasonal water table, generation of hazardous leachate can be controlled with a sheet piling cut-off. As with slurry walls and grout curtains, sheet piling cut-offs can form barriers that will redirect groundwater around or below the deposited wastes.

4.1.3.3 Design and Construction Considerations

For construction of a sheet piling cut-off, the pilings are assembled at their edge interlocks before they are driven into the ground. This is to ensure that earth materials and added pressures will not prevent a good lock between piles. The piles are then driven a few feet at a time over the entire length of the wall. This process is repeated until the piles are all driven to the desired depth.

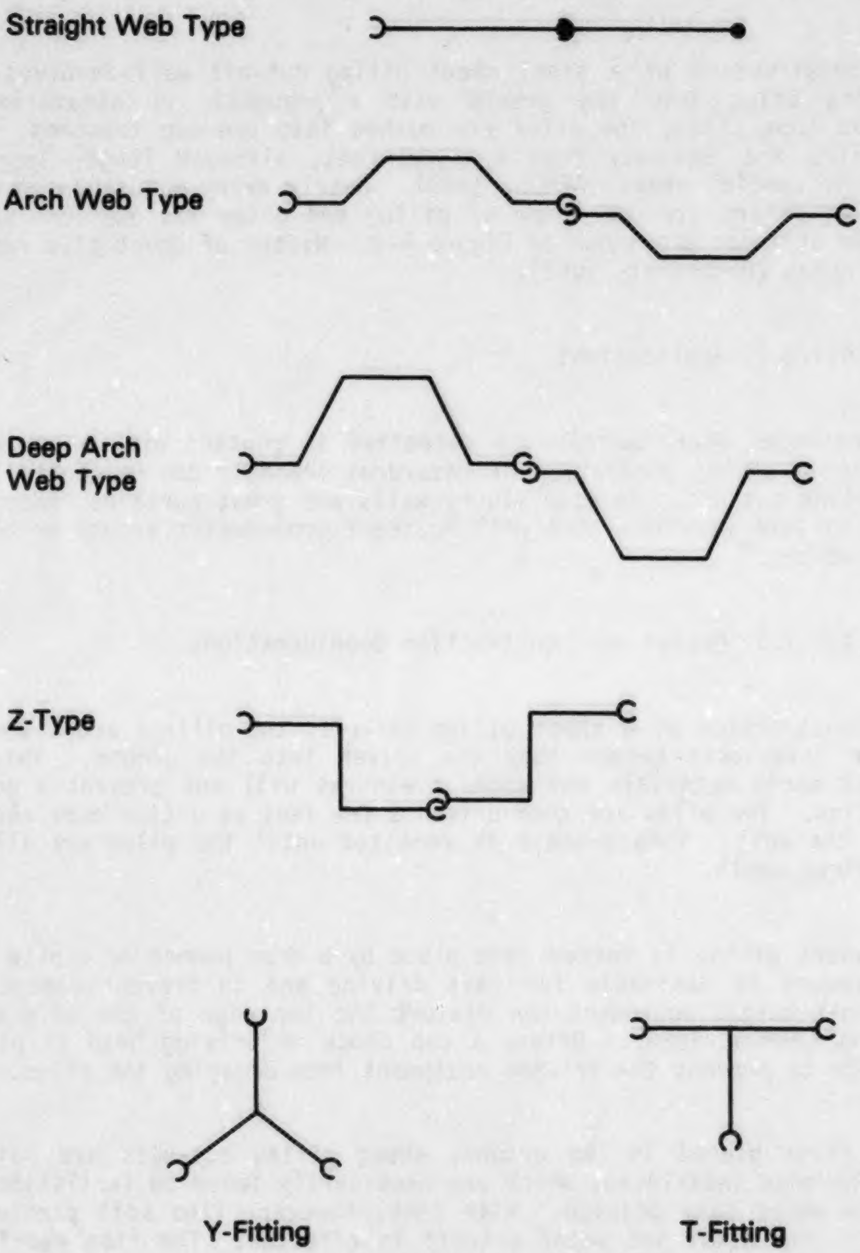
The sheet piling is forced into place by a drop hammer or a pile driver. Heavy equipment is desirable for fast driving and to prevent damage to the piles. Light-weight equipment can distort the top edge of the pile and slow the driving (ARMCO, 1980). Often, a cap block or driving head is placed on the top edge to prevent the driving equipment from damaging the piles.

When first placed in the ground, sheet piling cut-offs are quite permeable. The edge interlocks, which are necessarily loose to facilitate placement, allow water easy passage. With time, however, fine soil particles are washed into the seams and water cut-off is effected. The time required for this sealing to take place depends on the rate of groundwater flow and the

FIGURE 4-3

SOME STEEL PILING SHAPES AND INTERLOCKS

(Source: Ueguhardt, 1962)


Straight Web Type
Arch Web Type
Deep Arch Web Type
Z-Type
Y-Fitting
T-Fitting

texture of the soil involved. In very coarse, sandy soils, the wall may never seal. In such cases, it is possible to grout the piling seams, but this is a costly procedure.

The performance life of a sheet piling wall can be between 7 and 40 years, depending on the condition of the soil in which the wall is installed. Sheet piling walls have been installed in various types of soils ranging from well-drained sand to impervious clay, with soil resistivities ranging from 300 ohm cm to 50,000 ohm cm, and with soil pH ranging from 2.3 to 8.6. Inspections of these installed walls did not reveal any significant deterioration of the structure due to soil corrosion (EPA, 1978). Additional protection of the sheet piling wall against corrosion can be achieved by using hot-dip galvanized or polymer-coated sheet. Cathodic protection has also been suggested for submerged piling (EPA, 1978).

Steel sheet piles should not be considered for use in very rocky soils. Even if enough force can be exerted to push the piles around or through cobbles and boulders, the damage to the piles would be likely to render the wall ineffective.

4.1.3.4 Advantages and Disadvantages

The advantages and disadvantages of the sheet piling cut-off walls are summarized in Table 4-5.

TABLE 4-5

ADVANTAGES AND DISADVANTAGES OF THE SHEET PILING WALL

Advantages	Disadvantages
● Easy to install and readily available	● Cannot be used effectively in rocky soils
● Relatively inexpensive	● The wall initially is not waterproof
● Low maintenance requirements	● Corrosion of the wall may occur if exotic chemical substances exist in the soil

4.1.3.5 Costs

The unit costs of installing a sheet piling cut-off wall are summarized in Table 4-6.

TABLE 4-6

UNIT COST FOR INSTALLATION OF A SHEET PILING WALL

	Assumptions	Costs
Sheet Piling	Black steel	\$1,157/ton[1]
	Hot dipped galvanized steel (5 gage dimensions: 19.6 in. laying width, 3.18 in. front to back, and 20 ft. long)	\$1,317/ton[1]
	Installation	\$235/ton[2]

---

[1] ARMCO, 1980; Material and transportation costs.
[2] McMahon and Pereira, 1979.

As an example, the total cost of installing a sheet piling cutoff wall 1,000 feet long and 20 feet deep made of 5-gage galvanized steel sheet can be calculated as follows:

- The total area of the wall is:

$$(1{,}000\ \text{ft})\ (20\ \text{ft}) = 20{,}000\ \text{ft}^2$$

As a rule-of-thumb in the construction business, the adjusted area of the sheet piling wall is calculated by multiplying the required area by a factor of 1.6 to account for the area of the interlocking devices (Staples, 1980).

$$A_{\text{adjusted}} = A_{\text{required}} \times 1.6$$

$$(20{,}000\ \text{ft}^2)(1.6) = 32{,}000\ \text{ft}^2$$

According to ARMCO, the weight of 5-gage galvanized ARMCO steel sheet per unit of area is 11.6 lbs/sq. ft. Thus the total tonnage of sheet piling needed is:

$$(32{,}000\ \text{ft}^2)(11.6\ \text{lbs/ft}^2) = 371{,}000\ \text{lbs or } 186\ \text{tons}$$

- As shown in Table 4-6, the cost of 1 ton of sheet piling is $1,317; therefore, the material cost of the wall amounts to:

  $$(1{,}317/\text{ton})\ (186/\text{ton}) = \$245{,}000$$

- The installation unit cost of the sheet piling wall is also shown in Table 4-6. Therefore, the installation cost can be calculated as follows:

  $$(\$235/\text{ton})(186/\text{ton}) = \$43{,}700$$

- Therefore, the total cost of installing a sheet piling cutoff wall can be calculated as follows:

  (Materials Cost) + (Installation Cost) = Total Cost

  $$\$245{,}000 + \$43{,}700 = \$288{,}700$$

4.2 PERMEABLE TREATMENT BEDS

4.2.1 General Description

In many cases where a disposal site is located near the water table, groundwater may be contaminated with leachate from the site. In many of these cases, the groundwater is found at relatively shallow depths. Contaminated groundwater can be contained by constructing a physical barrier to prevent its flow, or it can be treated in place by constructing a permeable treatment bed that can physically and chemically remove the contaminants. Permeable treatment beds may become saturated or plugged in time, hence they may need replacement. They should, therefore, be considered as temporary rather than permanent remedial actions.

This section discusses possible methods for treating contaminated groundwater by using different types of permeable beds. These methods may have considerable potential for reducing the quantities of contaminants present in groundwater. The present state-of-the-art in this area of technology, however, is more or less conceptual.

4.2.2 Applications

Relatively few materials can be feasibly employed in permeable beds to control contaminated groundwater. These materials include:

- Limestone or crushed shell
- Activated carbon
- Glauconitic greensands or zeolite
- Synthetic ion exchange resins.

A limestone bed may be applied in cases where neutralization of acidic groundwater flow is needed. Limestone beds have also been claimed to be effective in removing certain metals such as cadmium, iron, and chromium (EPA, 1978). Crushed shell, which has the same chemical characteristics as limestone, can also be used as a material for permeable treatment beds. In most coastal regions where the availability of shells is good, crushed shell can become a very useful material for permeable treatment beds, and may compete with limestone in the control of acidic groundwater by this method.

In cases where the groundwater is contaminated with organic compounds, activated carbon may be applicable; however, costs of this material may be prohibitively high.

Glauconitic greensand deposits of the Atlantic Coastal Plain have been reported to exhibit good adsorption properties for several heavy metals (Spoljaric and Crawford, 1979). They are accessible as subsurface deposits in extensive areas of New Jersey, Delaware, and Maryland. The locations of these deposits suggest that permeable treatment beds using local greensand deposits might provide an economical remedy for contaminated groundwater at disposal sites in the Mid-Atlantic region.

Other materials that could be used for removing contaminants in groundwater are zeolite and synthetic ion exchange resins. These materials are very effective in the removal of heavy metal contaminants but are economically and practically infeasible for permeable beds because of problems such as short life, high cost, and re-activation difficulties. Therefore, these materials are not recommended for use except where engineering and economic evaluations prove their desirability in specific cases.

4.2.3 Design and Construction Considerations

The construction of a permeable treatment bed is essentially the same for all the above-mentioned materials. It consists of excavating a trench to intercept the flow of the contaminated groundwater, filling the trench with the appropriate materials, and capping the trench. Figure 4-4 shows a typical permeable treatment bed installation.

An important consideration in the design of a permeable treatment bed is the size and shape of the trench as it relates to the specific problem of groundwater contamination. The trench should be long enough to contain the plume of the contaminated flow and deep enough to stop the groundwater from flowing underneath. For practical design purposes, the depth of the bed is the distance between the ground surface and the bedrock or other impermeable layer (such as a layer of clay). The width of the trench is determined by the velocity of the groundwater flow, the permeability of the material used for treatment, and the contact time required for effective treatment. Further discussion on the determination of width for the trench is presented in later sections.

Trench excavation will require the use of conventional shoring such as sheet piling, and bracers and struts. Since the trench will intersect the water table, dewatering will be required during excavation. Groundwater pumped from the trench will likely be contaminated and therefore will probably require treatment.

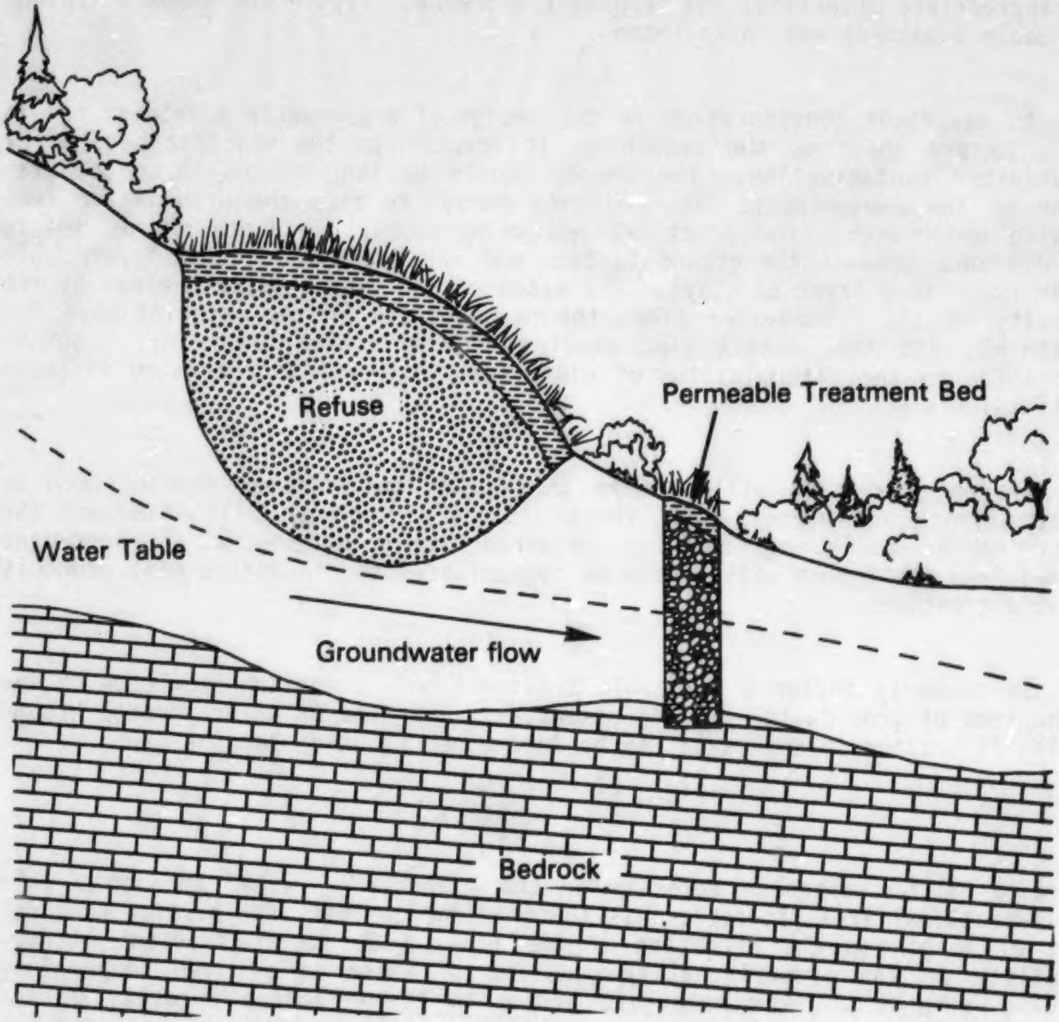
To properly design a permeable treatment bed, a working knowledge of the mechanisms of groundwater flow is essential. The flow of water through ground or its filtration through sand may be determined by using Darcy's law:

$$v = \frac{kI}{n}$$

where $\underline{v}$ is the approach velocity or the quantity of water through a unit cross-sectional area (ft/sec), I is the gradient or loss of elevation head per unit of length in the direction of the flow, $\underline{k}$ is the coefficient of permeability or the proportionality constant of water at a given temperature flowing through a given material, and $\underline{n}$ is the effective porosity of the material. Thus, by knowing the permeability coefficient of the soil through which the groundwater flows and the difference in hydraulic head between two points, one may be able to estimate the velocity of the flow or the volumetric flow rate of groundwater. Darcy's law is applicable to a steady flow of groundwater in a confined aquifer of uniform thickness, which is called a steady one-directional flow. In most cases, this ideal situation does not occur. However, Darcy's law can still be used to provide fair approximations of groundwater flow rate in field applications.

FIGURE 4-4

INSTALLATION OF A PERMEABLE TREATMENT BED


Refuse
Permeable Treatment Bed
Water Table
Groundwater flow
Bedrock

The thickness of the permeable treatment bed is a function of required residence time and the flow velocity of the groundwater through the bed, as expressed in the following equation:

$$w_b = (v_b)\ (t_c)$$

where w_b is the width or thickness of the bed, v_b the velocity of groundwater through the bed (ft/sec), and t_c the contact time (sec). To determine the thickness of the bed, groundwater flow velocity and contact time should be known.

For conservatism, the highest groundwater velocity found in the immediate area should be used as the design velocity of the flow through the bed. The bed thickness can then be estimated, using the flow velocity of groundwater through the bed and the optimum contact time (discussed below).

To insure that disturbance of the groundwater flow by the treatment bed is minimized, flow velocities within the soil and within the bed should be equal. Assuming that the hydraulic gradients in the bed and in the soil are the same, the permeability of the bed must be the same as the permeability of the soil. These permeabilities can be easily determined in the laboratory by using a "falling-head permeameter" (Fair et al., 1966; Johnson Division, 1975).

Residence time or contact time is a function of the level of contamination of groundwater and the treatment characteristics of the material used for the decontamination process. The residence time of the contaminated groundwater flow through the bed must be sufficient to insure optimum treatment conditions. Thus the determination of the optimum contact time, t_cm, requires a knowledge of the chemstry of the contaminated groundwater and of its reaction to treatment by the material that will be present in the bed.

To summarize, in the actual design of a treatment bed, the permeability of the soil and the hydraulic gradients in the field must be studied in order to estimate the actual velocities of the groundwater flow in the soil. Then the highest groundwater flow velocity is chosen as the design velocity of the flow through the bed, to insure optimum operational conditions. To obtain the same velocity of groundwater through the bed as through the soil, the permeability of the bed must be the same as the permeability of the soil. Consequently, the particle size of the material used in the bed must be chosen to achieve the desired permeability of the bed. This can be accomplished by conducting a series of bench-scale permeability tests with bed materials of different particle size. To design an effective treatment bed, an optimum contact time must be determined on the the basis of expected rates of interaction between the groundwater and the bed material. When the velocity of the flow through the bed and the optimum contact time are estimated, the bed thickness can be determined.

4.2.3.1 Limestone Bed

As previously discussed in Section 4.2.2, limestone may be used for the neutralization of slightly acidic groundwater. To a certain extent, it also may be used to remove metallic contaminants from groundwater.

Fuller and other researchers (EPA, 1978) have discussed the use of crushed limestone as an effective low-cost landfill liner to aid in the attenuation of the migration of certain heavy metals from solid waste leachates. According to the authors, dolomitic limestone (containing significant amounts of magnesium carbonate) is less effective in removing ions than purer limestone containing little magnesium carbonate. Therefore, in the design of a limestone treatment bed, it is recommended that limestone with high calcium content be used to remove heavy metals and to neutralize contaminated groundwater.

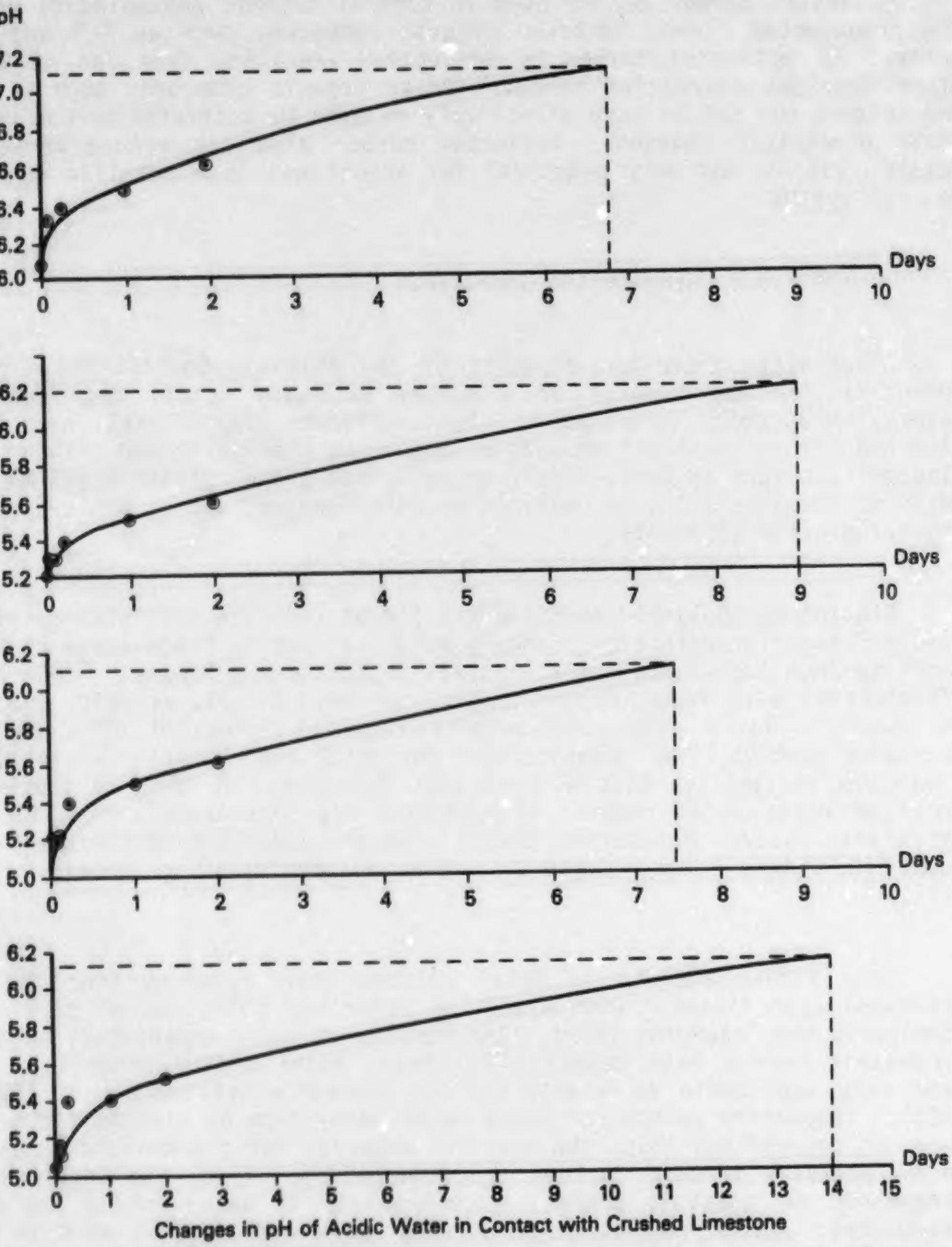
The particle size of limestone used depends on the results of the analysis of the type of soil in which groundwater flows and the level of groundwater contamination. The grade of limestone can vary from "gravel size" to "sand size." For practical design purposes, it is advisable to use a mixture of "gravel size" and "sand size" limestone to minimize settling of the bed as limestone dissolves. The smaller "sand size" particles help prevent excessive channelling through the bed and also improve the contact between the contaminated flow and the bed.

Fuller (EPA, 1978) studied the neutralization effect on acidic water flowing through a crushed limestone bed. The results, illustrated in Figure 4-5, show the relationship between changes in pH and the contact time. Thus, as shown in extrapolating the data present in Figure 4-5, the contact time needed to change 1 pH unit is in the range of 8 to 15 days.

Little information is presently available on determining the contact time needed for optimum removal of heavy metals. According to Fuller (EPA, 1978), the efficiency of limestone in removing heavy metals from leachate depends heavily on (a) contact time, (b) leachate concentration and (c) leachate pH. Limestone has been proved to be very effective in the attenuation of the migration rate of chromate, but an explanation for the mechanism of removal of this metallic is not available (EPA, 1978). Removal of metallic cations such as Be, Cd, Ni, and Zn from leachate were also studied, but no conclusive result was obtained. Thus more studies are needed to determine the effectiveness of limestone in the removal of heavy metals from landfill leachate and to determine the optimum contact time for the removal process.

FIGURE 4-5

pH CHANGES WITH CONTACT TIME



(Source: EPA, 1978)

4.2.3.2 Activated Carbon

Activated carbon may be used to control organic contaminants present in the groundwater flow. Nonpolar organic compounds such as PCB and CCl are removed by activated carbon by adsorption resulting from Van de Waals and other chemical attractive forces. Polar organic compounds such as alcohols and ketones may not be very effectively removed by activated carbon because of their electrical charges. Activated carbon also can remove certain heavy metals, but is not very practical for actual use in a metallic contaminants removal system.

4.2.3.3 Glauconitic Greensands

Glauconitic greensand deposits of the Atlantic Coastal Plain have high potential for the removal of a number of heavy metals from contaminated waters. Glauconite is a hydrous aluminosilicate clay mineral, rich in ferric iron and with significant amounts of potassium (Spoljaric and Crawford, 1979). Glauconite occurs as dark, light, or yellowish-green pellets 0.g-1 mm long, as casts of fossil shells, as coatings on other grains, and as a clayey matrix in coarser-grained sediments.

Studies by Spoljaric and Crawford (1979) indicate that glauconitic greensand had superior retention of heavy metal cations in bench-scale studies with leachate from the Pigeon Point landfill in northern Delaware. Highest removal efficiencies were reported for copper, mercury, nickel, arsenic, and cadmium, as shown in Table 4-7. The authors reported increased efficiencies with increased contact time. Contact time for bench scale testing was estimated at 2 minutes, suggesting that with contact times used in the field being in the order of days, metal removal efficiencies may be extremely high for many of the metals listed (Spoljaric, 1980). The glauconitic sand treatment was also found to reduce odors, suggesting that adsorption of organics occurred as well.

Only minute amounts of metal cations were released from the charged greensand upon flushing with distilled water and solutions of pH 2 and pH 12 (Spoljaric and Crawford, 1979). The results of these experiments suggest that greensands have a high capacity for heavy metal cation retention, and thus seem very applicable as a material for permeable treatment beds (Spoljaric, 1980). Saturation points for heavy metal adsorption by glauconitic greensands were not determined; thus, the sorptive capacity for glauconitic sands has yet to be assessed through further experimentation. Also, the applicability of greensands in treating higher concentrations of heavy metals has not been determined. While they appear promising, more experimental work is required before glauconitic greensands can be thoroughly assessed as a permeable treatment bed material.

TABLE 4-7

RESULTS OF CHEMICAL ANALYSES OF GREENSAND FILTRATION OF PIGEON POINT LANDFILL LEACHATE[1]

Cation	Cation concentration (ppm) Before filtration	After greensand	Percent retained by greensand
Aluminum	0.68	0.17	75
Arsenic (ppb)[2]	2.2	0.2	91
Cadmium (ppb)[2]	6	1	83
Calcium	129	48	63
Chromium	0.03	0.01	66
Cobalt	0.015	0.003	80
Copper	0.38	n.d.[3]	100
Iron	8.1	1.1	86
Lead	0.13	n.d.[3]	100
Magnesium	62	20	67
Manganese	4.1	0.48	88
Mercury (ppb)[2]	8.7	n.d.[3]	100
Nickel	0.074	0.003	96
Potassium	122	74	39
Silver (ppb)[2]	1.4	0.7	50
Na	275	175	36
Zinc	0.49	0.16	67
pH	7.65	6.29	

[1] Flow rate: 100 ml/min.
[2] Parts per billion (μg/liter).
[3] Not detected.
Source: Spoljaric and Crawford, 1979.

4.2.4 Advantages and Disadvantages of the Use of Permeable Treatment Bed

The advantages and disadvantages of the use of limestone beds, activated carbon beds, and glauconitic greensand beds are summarized in Tables 4-8, 4-9, and 4-10 respectively.

TABLE 4-8

ADVANTAGES AND DISADVANTAGES OF CRUSHED LIMESTONE TREATMENT BED

Advantages	Disadvantages
● Can be used to neutralize a slightly acidic groundwater stream	● Cementation or solidification of the limestone bed may occur, leading to plugging of the flow
● Applicable for the removal of certain heavy metals contained in groundwater	● Not effective for the removal of organic contaminants
● Good potential for successful control for chromate anion present in groundwater flow	● Not effective for the removal of organic contaminants
● Very cost effective to install since limestone is inexpensive and readily available	● Solution-channelling through bed may occur

TABLE 4-9

ADVANTAGES AND DISADVANTAGES OF ACTIVATED CARBON TREATMENT BED

Advantages	Disadvantages
● Very effective in the removal of nonpolar organic compounds from the groundwater flow	● Plugging of the bed may occur
● Readily available and easy to handle and install	● Not very effective for the removal of polar organic compounds
	● Presence of other chemicals in the groundwater may decrease the effectiveness of bed absorption

--continued--

TABLE 4-9 (Continued)

Advantages	Disadvantages
	● Desorption of the hazardous absorbed materials to the clean water flow may occur, resulting in recontamination
	● Removal and disposal of spent activated carbon is difficult and hazardous
	● Cost of the material is very high
	● Competitive absorption with large organic molecules may decrease the removal effectiveness of the bed
	● Life of the bed may be very short in the presence of complex organic compounds such as humic compounds

4.2.5 Cost

The cost of installing a permeable treatment bed includes the cost of trench excavation and dewatering, trench shoring, bed materials, and bed installation. The unit costs for these operations are summarized in Table 4-11.

TABLE 4-10

ADVANTAGES AND DISADVANTAGES OF GLAUCONITIC TREATMENT BED

Advantages	Disadvantages
● Apparent high effectiveness in the removal of many heavy metals	● Saturation characteristics unknown
● Good residence time characteristics for efficient treatment; relatively little material required for bed	● Area of application probably limited by transportation costs to Mid-Atlantic region
● Abundant in New Jersey, Delaware, and Maryland	● May require land purchase since it does not seem to be commercially mined
● Good metal retention characteristics	● Reduction in permeability and plugging of bed may occur after a time
● Good permeability	● May reduce pH
	● Removal efficiencies of metals at high concentrations unknown

TABLE 4-11

UNIT COSTS FOR INSTALLATION OF A PERMEABLE TREATMENT BED

	Assumptions	Costs
Trench excavation	20 ft deep, 4 ft wide, by backhoe	\$1/cubic yard[1]
Spreading	Spread nearby to grade trench and cover	\$0.66/cubic yard[1]
Well-point dewatering	500 ft header 8" diameter, for one month	\$75/linear foot[1]

--continued--

TABLE 4-11 (Continued)

	Assumptions	Costs
Sheet piling	Pull and salvage	$5.70/square foot[1]
Walers, connections, struts	2/3 salvage	$105/ton[2]
Limestone	Mixed "gravel size" and "sand size"	$4.75 - $6.00/ton or $6.75 - $8.00/ cubic yard[5]
	Installation	$1.90/cubic yard[1]
	Total cost including overhead and profit	$10 - $11.5/cubic yard[1]
Activated carbon	Coarse size	$0.5/lb[6] or $1,180/cubic yard
	Installation	$1.90/cubic yard[1]
	Total cost including overhead and profit	$1,360/cubic yard[1]

---

[1] Godfrey, 1980; Costs are total, including contractor overhead and profit.
[2] Godfrey, 1980; Materials only.
[3] Milam, 1980; Assumed 50 percent installation cost.
[4] Universal Linings, 1980.
[5] Germany Valley Limestone Co., West Virginia, 1980; materials only.
W.S. Frey Co., Virginia, 1980; materials only.
McDonough Bros., Inc., Texas, 1980; materials only.
[6] Calgon, 1980; materials only.

These costs are based on the following assumptions:

- The trench dimensions are 20 ft deep, 4 ft wide, and 1,000 ft long.
- Limestone used for the treatment bed is a mixture of "gravel size" and "sand size" particles.
- The average bulk density of the limestone mixture is 100 lb/ft^3.
- The average particle density of activated carbon is 1.4 g/cc or 87.4 lb/ft^3 (EPA, 1978).

Costs of glauconitic greensand would entail possible land purchase, borrow excavation, and hauling to the disposal site. Costs for buying bulk quantities of glauconitic greensand from a producer were not available at the time of this writing. The cost of zeolite and synthetic ion exchange resins is very expensive (about \$5/lb), and thus might prove to be economically infeasible.

As an example, the total cost of a permeable treatment bed 1,000 feet long, 4 feet wide and 20 feet deep can be calculated as follows:

- The total volume of the treatment bed is:

  (1,000)(4 ft) (20 ft) = 80,000 ft^3 or 3,000 cubic yard.

  The surface area of the side of the trench is:

  (1,000 ft)(20 ft) = 20,000 ft^2

- The unit costs for trench excavation and preparation are shown in Table 4-11. The total cost of trench excavation and associated works includes the following costs:

  - Trench excavation cost:

    (\$1/cubic yard)(3,000 cubic yard) = \$3,000

  - Spreading cost:

    (\$0.66/cubic yard)(3,000 cubic yard) = \$1,980)

  - Well point dewatering cost:

    Assuming that two 8"-diameter 500-ft headers are needed.

    (\$75/lin. ft)(500 ft)(2) = \$75,000

  - Sheet piling cost:

As a rule-of-thumb in the construction business, the total area of sheet piling needed can be estimated by multiplying the side areas of the trench by a factor of 1.6 to account for the allowance area for the interlocking devices (Staples, G., JRB Associates , 1980). Thus the total area of sheet piling required is:

$$(2)(20,000\ at^2)(1.6) = 64,000\ ft^2$$

According to ARMCO, The average weight of sheet piling per square foot of wall is 10 lbs. Thus the total tonnage of the sheet piling can be calculated as follows:

$$(64{,}000\ ft^2)(10\ lbs/ft^2) = 640{,}000\ lbs\ or\ 290\ metric\ tons.$$

The cost of the sheet piling can then be calculated as follows:

$$(64{,}000\ ft^2)(\$5.70/ft^2) = \$365{,}000$$

Waler, connection, and strut costs:

As a rule-of-thumb in the construction business, the total tonnage of walers, connections, and struts are 20 percent of the weight of sheet piling (Staples, 1980). Therefore, the tonnage required for walers and struts is:

(290 tons)(0.2) = 58 tons

The cost for walers and struts is:

($105/ton)(58 ton) = $6,100

Total cost of trench construction:

$3,000 + $1,980 + $75,000 + $365,000 + $6,100 = $451,000

The total cost of a permeable treatment bed includes trench construction and the costs of materials and their installation. Unit costs for the materials and their installation are given in Table 4-11.

In the case of a limestone treatment bed, materials and their installation costs are:

($11.5/cubic yard)(3,000 cubic yard) = $34,500

The total cost of the limestone bed is:

(Trench construction cost) + (Materials and installation cost) =

Total Cost

$451,000 + $34,500 = $485,500

The total cost of the installation of an activated carbon bed can be estimated as follows:

Materials and installation cost:

($1,360/cubic yard)(3,000 cubic yard) = $4,080,000

Therefore the total cost of the activated carbon bed is:

$451,000 + $4,080,000 = $4,531,000

4.3 GROUNDWATER PUMPING

This section describes several applications of groundwater pumping to control contaminated groundwater beneath a disposal site. These approaches emphasize the active diversion of groundwater as compared to passive approaches of installing either impermeable barriers or permeable treatment beds. Three applications are described:

- Pumping to lower a water table
- Pumping to contain a plume
- Groundwater treatment systems

4.3.1 Water Table Adjustment

4.3.1.1 General Description

Groundwater pumping can be used to lower the water table in a particular area. By placing wells close together, the combined cones of depressions of each well can result in a depression network, in which the effective elevation of the groundwater has been lowered. Pumping to cause a change in groundwater elevation has a number of applications, which are discussed below.

4.3.1.2 Applications

Groundwater pumping to lower the water table may be a suitable remedial action for contaminated groundwater under several conditions. Specific applications include:

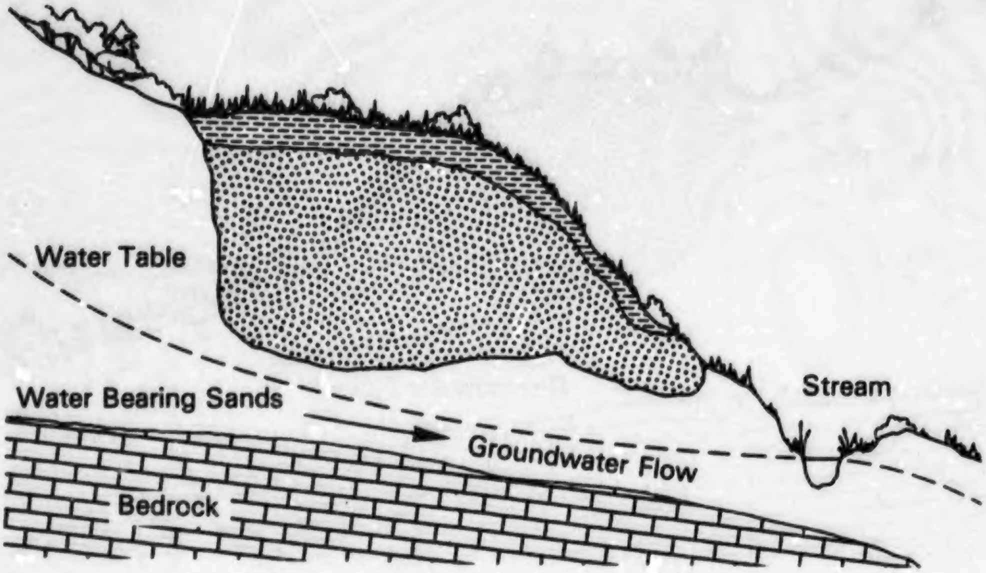
- Lowering an unconfined aquifer sufficiently so that contaminated groundwater does not discharge to a receiving stream that is hydraulically connected
- Lowering the water table so that it is not in direct contact with the waste site
- Lowering the water table to prevent leaky aquifers from contaminating other aquifers

These three applications are illustrated in Figures 4-6 through 4-8. A groundwater treatment system will frequently be needed to treat the water after pumping. (See Section 4.3.3.)

FIGURE 4-6

LOWERING A WATER TABLE TO PREVENT STREAM DISCHARGE OF CONTAMINATED WATER

Before Pumping


Water Table
Stream
Water Bearing Sands
Groundwater Flow
Bedrock

After Pumping

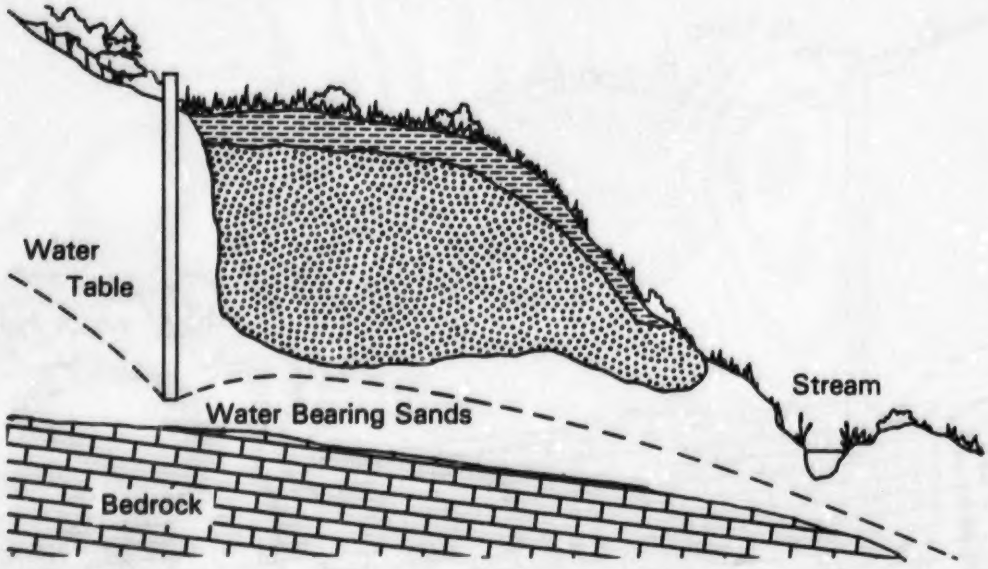
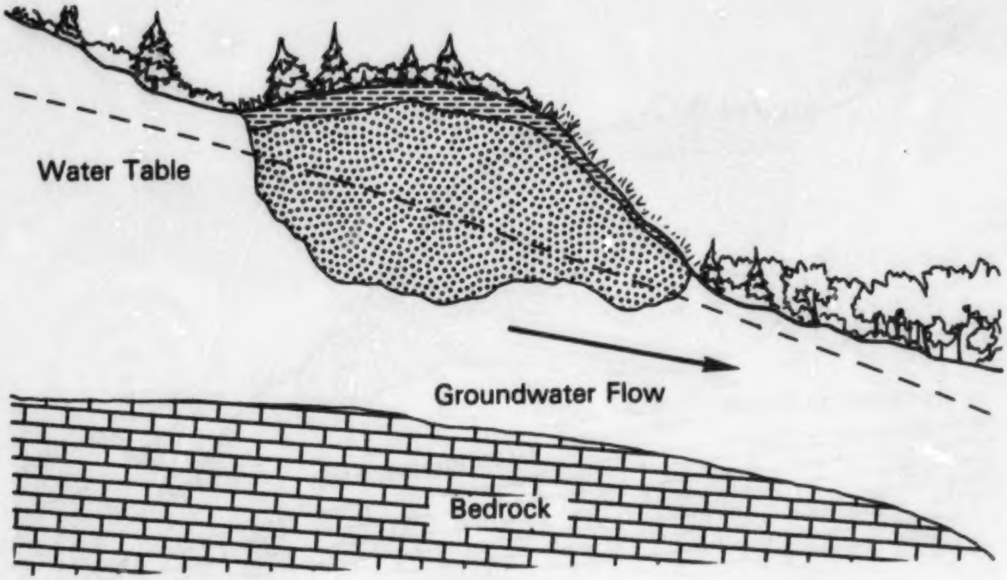

Water
Table
Stream
Water Bearing Sands
Bedrock

FIGURE 4-7

LOWERING THE WATER TABLE TO ELIMINATE CONTACT WITH A DISPOSAL SITE

Before Pumping


Water Table
Groundwater Flow
Bedrock

After Pumping

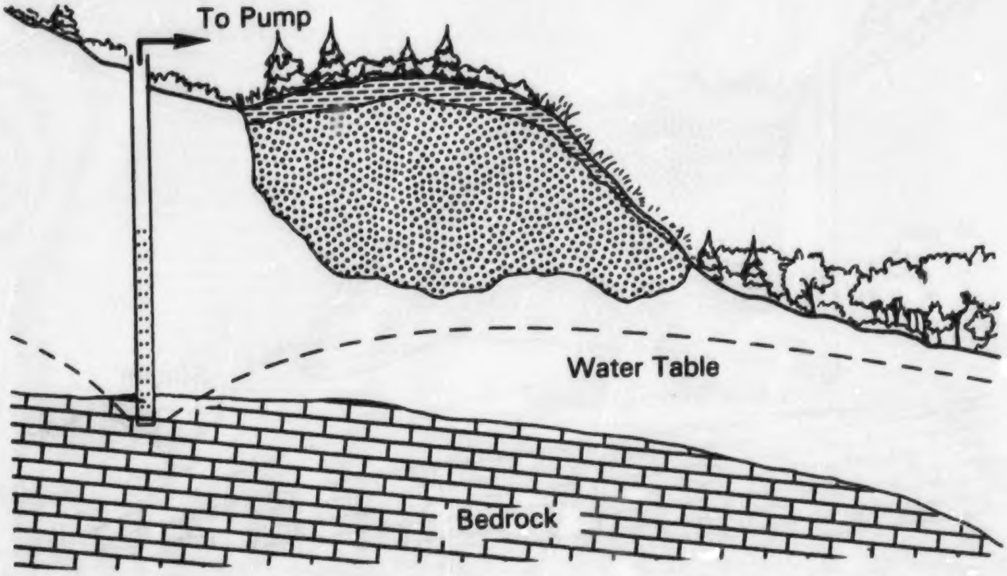
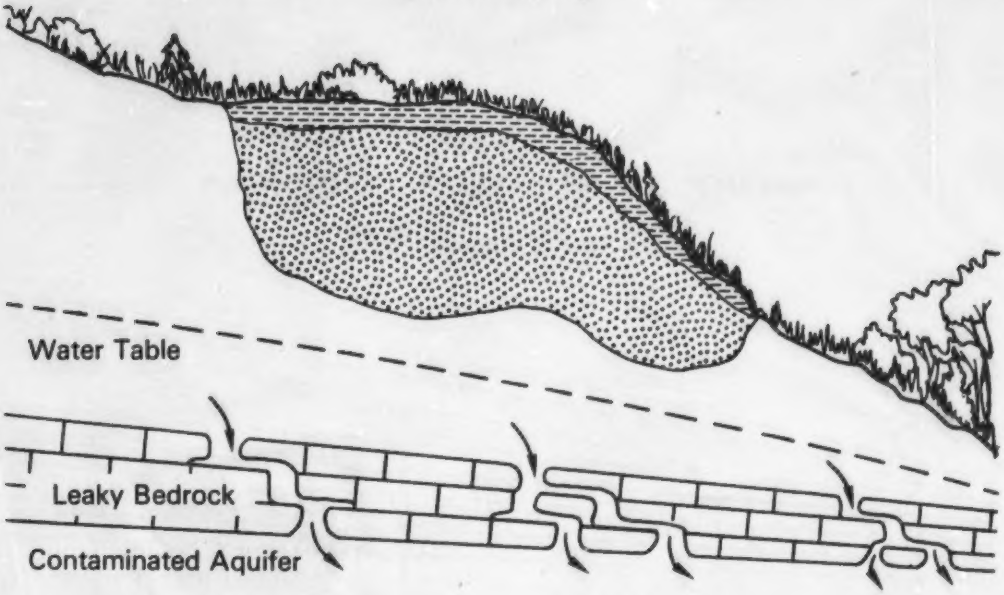

To Pump
Water Table
Bedrock

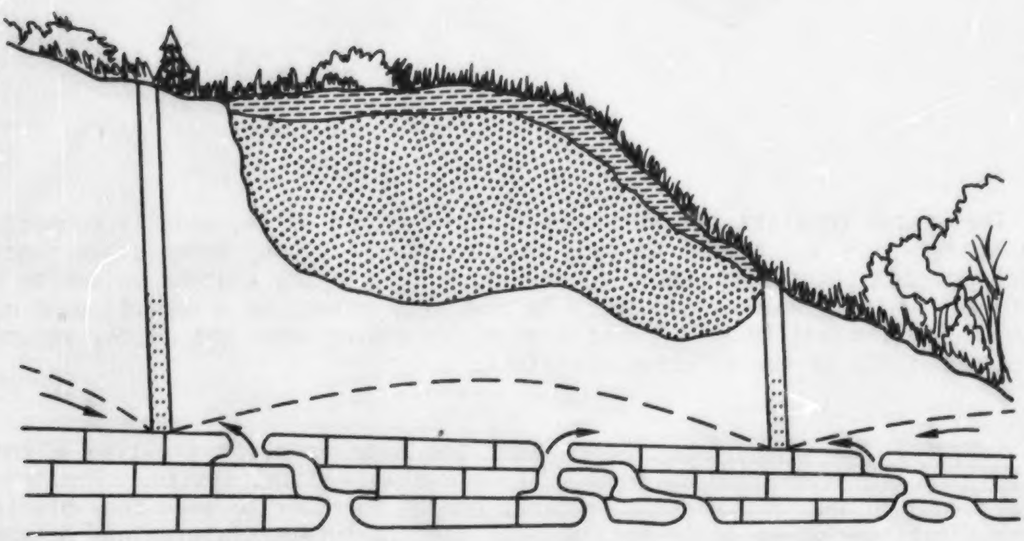
FIGURE 4-8

LOWERING WATER TABLE TO PREVENT CONTAMINATION OF AN UNDERLYING AQUIFER

Before Pumping


Water Table
Leaky Bedrock
Contaminated Aquifer

After Pumping



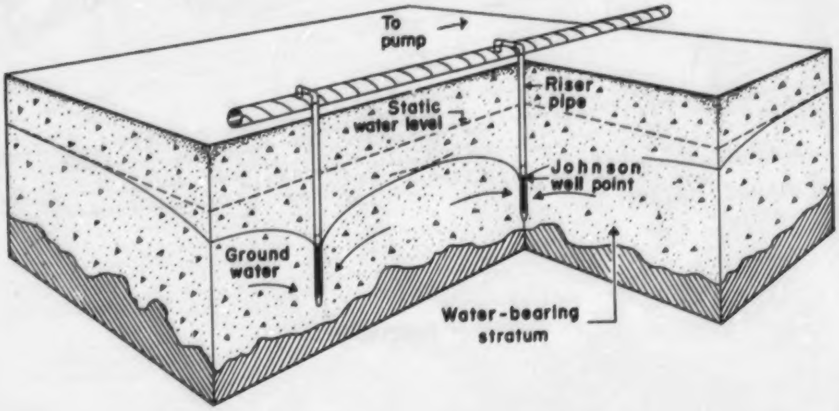
4.3.1.3 Design and Construction Considerations

The water table can be lowered by using a well point dewatering system or by using deep wells--a method discussed later in this section. Figure 4-9 shows a well point system.

FIGURE 4-9

SCHEMATIC OF A WELL POINT DEWATERING SYSTEM

(Source: Johnson, 1975)



The system consists of a group of closely spaced wells, usually connected by a header pipe and pumped by suction centrifugal pumps, submersible pumps, or jet ejector pumps, depending on the depth of pumping and the volume to be dewatered. A pump may be connected to one well point, or a central pump may be used for the entire well point system, depending upon the depth, volume, and permeability of the affected materials.

Lowering the groundwater level over the site involves creating a composite cone of depression by pumping from the well point system. The individual cones of depression must be close enough together so that they overlap and thus pull the water table down several feet at intermediate points between pairs of wells (Johnson Division, 1975).

The proper design and construction of wells and well points involves an understanding of the particulars of well hydraulics.

Darcy's Law tells us that flow in a porous medium varies directly with the characteristics of the medium and the hydraulic gradient. The expression for these relationships, taking into account the effects of porosity, is:

$$V = \frac{kI}{n}$$

Where:
k = Hydraulic conductivity (ft/day)
I = Hydraulic gradient (dimensionless)
V = Apparent velocity (ft/day)
n = Effective porosity (percent)

The hydraulic gradient (I) is the difference in hydraulic head divided by distance along the flow path in fluid flow. With increasing velocity, the hydraulic gradient increases as the flow converges towards a well. As a result, the lowered water surface develops a continually steeper slope towards the well. The form of this surface resembles a cone-shaped depression. The distance from the center of the well to the limit of this cone of depression is called the radius of influence. The hydraulic conductivity is measured using the meinzer unit, defined as the flow of water in gallons per day through a cross-sectional area of 1 square foot under a hydraulic gradient of 1 at a temperature of 60° F. The value of K depends upon the size and arrangement of the particles in an unconsolidated formation and the size and characteristics of the surfaces of crevices, fractures, or solution openings in a consolidated formation. Figure 4-10 shows typical hydraulic conductivities for various soil and rock types. Darcy's Law remains valid only under conditions of laminar flow, involving fluids with a density not significantly higher than pure water.

Two other factors, transmissivity (T) and storativity (S), also affect the rate of flow. The coefficient of transmissivity indicates how much water will move through a formation and is equivalent to the permeability times the saturation thickness of the aquifer. The coefficient of storativity indicates how much water can be removed by pumping and draining and is defined as the volume of water released from or taken into storage per unit area of aquifer per unit change in hydraulic head normal to the surface.

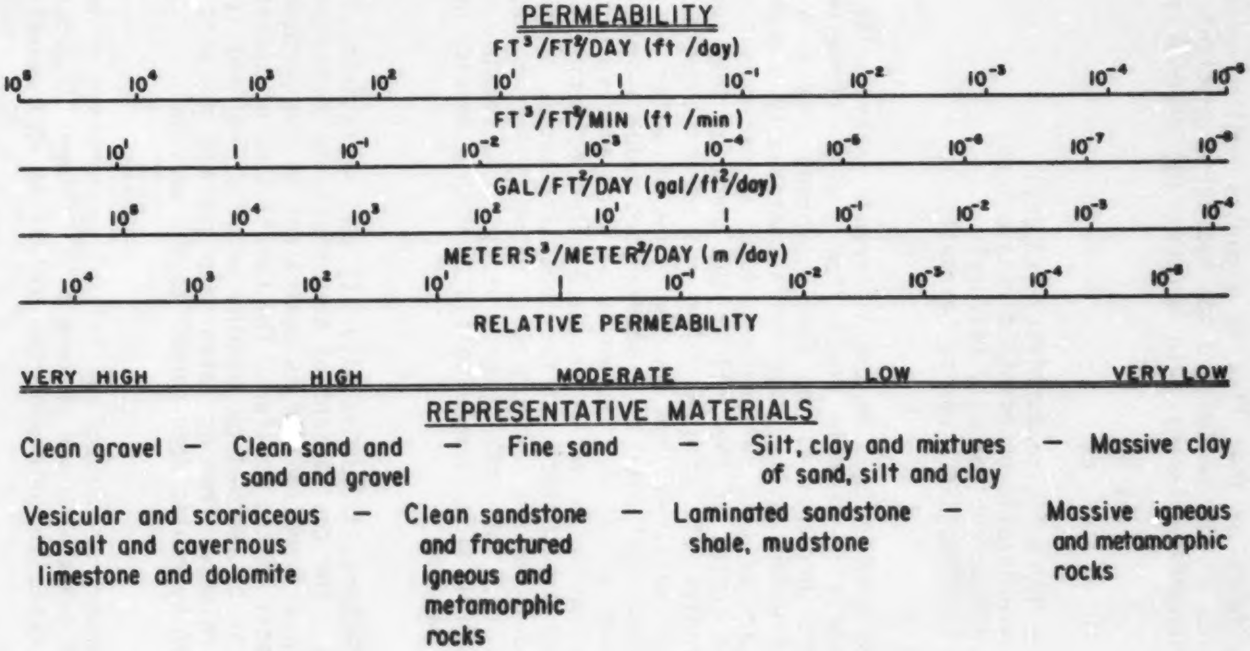
The usual procedure during the initial exploration of an aquifer is to drill a number of wells with one or more observational piezometers and then to conduct pumping tests to determine the values of transmissivity and storativity.

Once the aquifer properties of transmissivity (T) and storativity (S), have been determined, it is possible to predict the drawdown in hydraulic head

FIGURE 4-10

HYDRAULIC CONDUCTIVITIES OF SOIL AND ROCK

(Source: U.S. Department of Interior, 1977)



in a confined aquifer at a distance, r, from the well and at a time, t, for a given pumping rate (Q). Thus, by determining the drawdown at various radii from the well, one can determine the radius of influence for a given pumping rate.

For a given aquifer, the cone of depression initially increases in depth and extent with increasing pumping time until eventually it levels off. Drawdown at any point at a given time is directly proportional to the pumping rate and inversely proportional to aquifer transmissitivity and storativity. (Freeze and Cherry, 1979).

The Theis equation, used to estimate drawdown, is expressed as follows:

$$s = \frac{Q}{4\pi T} W(u)$$

Where:

s = Drawdown (ft)
Q = Pumping rate (gal/day)
$W(u)$ = Well function; dimensionless

$u = \frac{r_2 S}{4Tt}$
r = Radius from well (ft)
T = Transmissivity (gal/day/ft)
S = Storativity (dimensionless)

For a specific value of u, W(u) can be calculated from values shown in Table 4-12. As time approaches infinity, the value of W(u) levels off.

Figure 4-11 expresses the drawdown in potentimetric surface from a confined aquifer being pumped by two wells with equal flow.

For a system of n wells pumping at rates Q_1, Q_2 -- Q_n, drawdown at radius, r, from each well can be calculated as follows:

$$s = \frac{Q_1}{4\pi T} W(u_1) + \frac{Q_2}{4\pi T} W(u_2) + \text{--} + \frac{Q_n}{4\pi T} W(u_n)$$

TABLE 4-12

VALUES OF W(μ) FOR VARIOUS VALUES OF μ

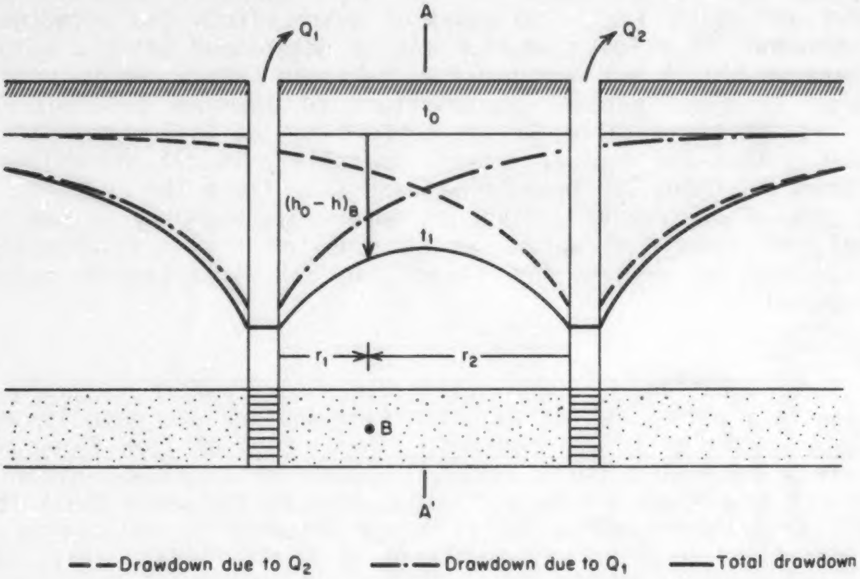
u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
x 1	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
x 10^{-1}	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
x 10^{-2}	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
x 10^{-3}	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
x 10^{-4}	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
x 10^{-5}	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
x 10^{-6}	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
x 10^{-6}	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
x 10^{-8}	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
x 10^{-9}	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
x 10^{-10}	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
x 10^{-11}	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
x 10^{-12}	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
x 10^{-13}	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
x 10^{-14}	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
x 10^{-15}	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

Source: Freeze, 1979
See Copyright Notice, page 496.

FIGURE 4-11

DRAWDOWN IN POTENTIOMETRIC SURFACE OF A CONFINED AQUIFER BEING PUMPED BY TWO WELLS

(Source: Freeze, 1979)
See Copyright Notice, Page 496



Theis's equation for drawdown estimates is based on an ideal aquifer, or one that is homogeneous and isotropic. In an isotropic aquifer, the transmissivity is independent of the direction of measurement.

In reality, drawdown at a given point and the total radius of influence for a well are determined by the following factors:

(1) Rate of pumpage

(2) Permeability and thickness of the water-bearing strata

(3) The manner in which groundwater is replenished

(4) Presence of boundaries that limit the extent of the aquifer

(5) Length of time that pumping continues (Johnson Division, 1975)

The Theis equation cannot account for factors 3 and 4 listed above, and numerous equations have been developed to predict drawdown considering these factors.

A basic assumption of Theis's equation, that geological formations overlying a confined aquifer are impermeable, is seldom true. As would be expected, drawdown in leaky aquifers is less than that in completely confined aquifers, and predictions based on the Theis equation therefore provide a conservative estimate; i.e., the equation overpredicts the drawdown. Estimates of drawdown in a leaky aquifer can be determined using a solution derived by Hantush (1960) and Neuman and Witherspoon (1969a, 1969b). The reader should refer to these papers for solutions to drawdown problems for leaky aquifers. Aquifers generally do not conform to the basic assumption of the theis equation that the aquifer extends infinitely in all directions. Definite geologic and hydraulic boundaries generally limit the aquifer. When an expanding cone of depression strikes an impervious boundary on one side of a pumped well, no additional water can be supplied from that direction. The cone must expand and deepen more rapidly in all directions to maintain the yield of the well.

The Theis equation is only applicable for drawdown estimates from a confined aquifer; estimates for an unconfined aquifer are complicated by the fact that, in addition to the horizontal components of flow found in a confined aquifer, there is also a vertical component of flow. Hydraulic gradients induced by pumpage create a drawdown cone in the water table itself and thus cause vertical components of flow. A solution to estimating drawdown from an unconfined aquifer was developed by Neuman (1972, 1973, 1975) and accounts for the effect of gravity drainage. The equation is similar to the Theis equations:

$$s = \frac{Q}{4\pi T} W(u_a, u_b, N)$$

where W $(u_a, u_b, N)$is the unconfined well function and N is the square of the radius over the square of the initial saturated thickness of the aquifer. W (u_a, u_b, N) can be determined from theoretical curves developed by Neuman (1975). This approach is similar to Boulton's method.

From the above equations, the drawdown at various distances from the well can be determined and a distance-drawdown diagram can be drawn. The well's radius of influence can be determined directly from these diagrams.

Designs of well point dewatering systems vary considerably, depending on the depth to which dewatering is required, the transmissivity and storativity of the aquifer, the size of the landfill, and the depth of the water-bearing formation.

- Depth, Spacing, and Sizing of Pipes

Depth for well points will be governed by the depth to which the water table must be lowered. This, in turn, influences the type of pump that can be used and the diameter of the well points and riser pipes. The maximum drawdown that can be maintained in the formation adjacent to each well is the depth of the vacumm or suction head, in feet, developed by the pumping equipment minus the distance from the center of the pump to the static water level and minus the head loss in the piping and well points themselves.

Where the landfill is situated in the water table, or where the depth to water-bearing sands is shallow, a well point system with a centrifugal suction pump located in the center of the header may be suitable. Theoretically, the maximum suction lift obtainable with suction pumps is about 20 to 25 feet, but friction losses reduce this to about 15 to 18 feet (Johnson Division, 1975). Therefore, a dewatering system using suction pumping should not be expected to pump more than 15 feet (Johnson Division, 1975; U.S. Department of Interior, 1977). Since the height of the landfill often is less around the periphery, considerable advantage would be gained by situating the well system along the periphery of the site to take advantage of the suction lift available. One potential problem with this approach is that the composite cones of depression may not encompass the center of the landfill. In this case, it may be necessary to include a few deep, high-capacity wells equipped with jet injector or submersible pumps in the center of the site.

Pipe size for well points is generally determined from experience and verified in the field for specific sites. For silt or other fine-grained materials, well points with a diameter of about 1.5 inches are generally satisfactory when centrifugal suction pumps are used, but the diameter required can be as much as 6 inches, depending upon the permeability of the soil. One-inch-diameter riser pipes are suitable for small-diameter well points; this size should be increased to 2.0 to 2.5 inches for well points with a diameter of 3.5 inches. (Johnson Division, 1975; U.S. Department of Interior, 1977.)

The length of well screens depends on the thickness of the aquifer and on the stratigraphic properties of the aquifer.

Where the depths to water-bearing sands are greater than 10 feet or so, where water-bearing sands must be dewatered to depths greater than 10 to 15 feet, or where several layers of stratified material must be penetrated, deep well point systems will probably be needed and submersible or jet ejector-based pumps should be used. Jet ejector pumps can lift water from 100 feet or more when connected to individual riser pipes. However, this type of system requires an additional header system to bring operating water to each pump. This results in higher operating costs. Submersible pumps have almost unlimited capacities, but are more costly. Although operation and maintenance costs for jet ejector pumps and submersible pumps are higher than for suction lift pumps, they will function effectively in deep wells (Johnson Division, 1975). Required piping diameter is larger for well-point dewatering systems pumped with jet ejector or submersible pumps. The diameter of well points may reach 6 inches for a jet ejector pump system with riser pipes of 3.5 inches or larger in order to permit installation of the jet ejector pump.

Well point spacing is based on the radius of influence of each well and the composite radii of influence needed to lower the water table. Once storativity and transmissivity have been determined, the equations discussed previously can be used to determine drawdown and area of influence. In practice, spacing for a few well points would be determined and then field-tested; any necessary adjustments would then be made to account for the fact that wells do not always meet the idealized conditions assumed in equations to estimate drawdown.

When centrifugal suction pumps are used, well points are usually spaced from 2 to 6 feet apart, depending on the permeability of the water-bearing materials and the depth to which the water table must be lowered. In silt and fine sands, 2.0 to 2.5 feet is the usual spacing. Where deep well points are required, and a jet ejector pump is used, spacing of 4 to 10 feet is common. No generalizations can be made for submersible pumps, since pumping capacities vary so widely (Johnson Division, 1975; U.S. Department of Interior, 1977).

- Installation

Well points are made to be driven in place, to be jetted down, or to be installed in open holes. The most common practice is to jet the well points down to the desired depth, to flush out the fines, leaving the coarser fraction of material to collect in the bottom of the hole, and then to drive the point into the coarser materials.

A method used in some unstable material consists of jetting down or otherwise sinking temporary casing into which the well point and riser pipe are installed. As the casing is pulled, gravel may be placed around the well point.

- Special Cases

In several instances, design modifications will be required or at least various methods should be compared for cost-effectiveness.

- Fine silts and other slowly permeable materials cannot be readily drained by well point systems alone. However, soils can be partially drained and stabilized by vacuum wells or well point systems that create negative pore pressure or tension in the soil. The well points should be gravel-packed from the bottom of the hole to within a few feet from the surface of the poorly permeable material. The remainder of the hole should be sealed with bentonite or other impermeable materials. If a vacuum is maintained in the well screen or pack, flow towards the well points is increased. Such a system usually requires closely spaced well points, and pumping capacity is reduced. Vacuum booster pumps may be required on the headers or individual wells for effective operation (Johnson Division, 1975).

- Vertical sand drains may be used in conjunction with well points to facilitate drainage in stratified soils. The drains, usually 16 to 20 inches in diameter, are installed on 6- to 10-foot centers through the impermeable layers that need to be dewatered and are extended to underlying permeable layers where well points are placed.

- Two or more well point systems may be required when two or more strata of water-bearing sand are separated by impermeable barriers. The depth for dewatering will be different for each system, and consequently pipe lengths and diameters and pumping requirements will be determined independently.

4.3.1.4 Advantages and Disadvantages

Advantages and disadvantages of well point pumping to adjust the water table are listed in Table 4-13.

4.3.1.5 Costs

Costs for equipment and construction of well point dewatering systems are listed in Table 4-14. Costs for pumping will vary widely with site conditions and pumping requirements. Costs shown in Table 4-14 should be considered only as examples. These unit costs have been used for cost estimates of a hypothetical site where dewatering was needed to lower a water table. It is assumed that the water table is to be lowered 12 feet along the perimeter of the site and that 2,500 feet of header pipe will be required.

Field testing and theoretical determinations using the Theis equation specified well point placement at 6-foot intervals. The system will require a centrifugal suction pump with a lift of 15 feet, 2-inch-diameter well points, and a 6-inch-diameter header. The well points will be placed around the periphery of the site. However, in order to obtain a composite cone of depression under the entire site, two high capacity wells were found to be needed within the site. These wells will be 30 feet deep and will require 4-inch submersible pumps and 6-inch well casing. Three monitoring wells will

TABLE 4-13

ADVANTAGES AND DISADVANTAGES OF WELL POINT SYSTEMS FOR WATER TABLE ADJUSTMENTS

Advantages	Disadvantages
● High design flexibility	● May not adequately drain fine silty soils, and flexibility is reduced in this medium
● Good on-site flexibility since the system can be easily dismantled	● Higher operation and maintenance costs than for artificial groundwater barriers
● Construction costs may be lower than for construction of artificial groundwater barriers	
● Good reliability when properly monitored	

also be required around the periphery of the site to determine the height of the water table. The total cost of equipment, construction, and installation was calculated to be $240,100. The calculations for these estimates are detailed below:

● 2,500 feet of 6-inch header pipe --(2,500 feet) ($35/linear foot)	= $ 87,500
● 2" -15' well points installed every 6'	
--(416 well points)(15')($22.6/ft.)	= $141,000
--(416 well points)($15 for fittings)	= 6,200
● Centrifugal suction pump (1 pump) ($284/pump)	= 284
● 2 High capacity wells	
--($2.50 per inch diameter per foot)(6 inches) (30 feet) (2 wells)	= 900
--(two 4-inch submersible pumps)($1,175)	= 2,350
● 6-inch PVC well casing --($6.50/ft.)(30 ft.)(2 wells)	= 390

- 3 monitoring wells to depths of 15 feet
  --($2.50 per inch diameter per foot)(4 inches)
  (15 feet)(3 wells) = 450
- 3 centrifugal pumps x $284/pump = 852
- Casing
  --(15 feet)(3 wells)($4.50/ft.) = 203

$240,129

4.3.2 Plume Containment

4.3.2.1 General Description

This section considers three applications of pumping in order to contain a plume:

1. Use of a series of extraction and injection wells that will allow water within the plume to be pumped, treated, and pumped back into the aquifer
2. Low rates of pumping to contain a plume with no subsequent recharge to the aquifer
3. Pumping and treatment of the plume followed by recharge using recharge basins

TABLE 4-14

UNIT COSTS FOR EQUIPMENT USED IN WELL POINT DEWATERING

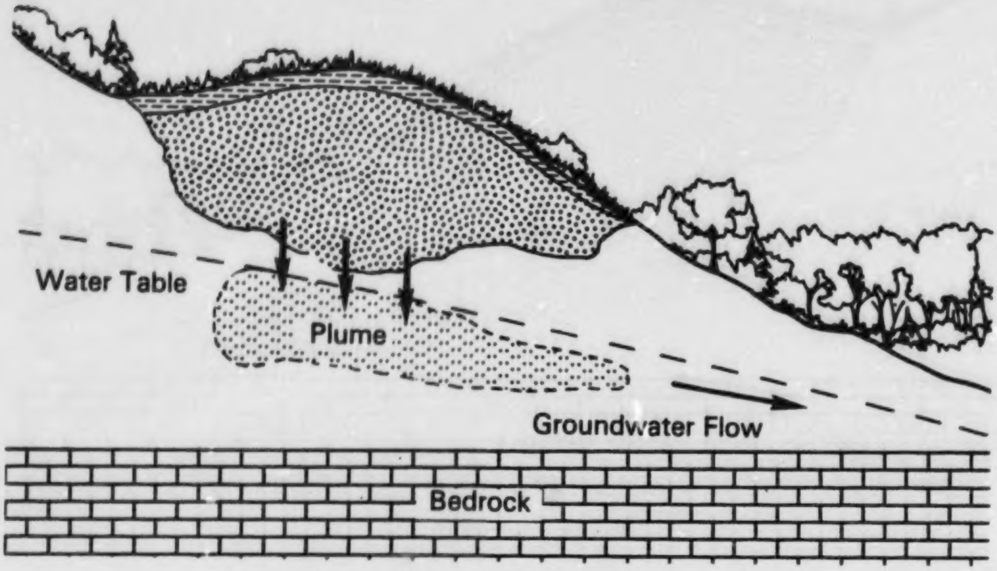
Unit	Cost	Source
2-inch wellpoint	$22.6/ft. + $15 for fittings	1
Header pipe - 6 inches	$35/ft.	2
4-inch wellpoint	$30/ft. + $28 for fittings	1
Header pipe - 8 inches	$46/ft.	2
Centrifugal suction pump 15-ft. lift; 4-5 GPM	$284	3
Jet ejector pump - 120 ft. lift; 5GPM; 3/4 H.P.	$530	2
4-inch submersible pump- 180 ft. lift; 23 GPM	$1,175	2
Monitoring Wells	$2.00 - 2.50 per inch diameter per foot of depth (without casing)	4
4-inch PVC casing 6-inch PVC casing	$4.50/ft. $6.50/ft.	5

[1] Davis, 1980
[2] Godfrey, 1980
[3] Jacuzzi Pumps, 1980
[4] U.S. Department of Interior, 1977
[5] Leazer Pumps and Wells, 1980

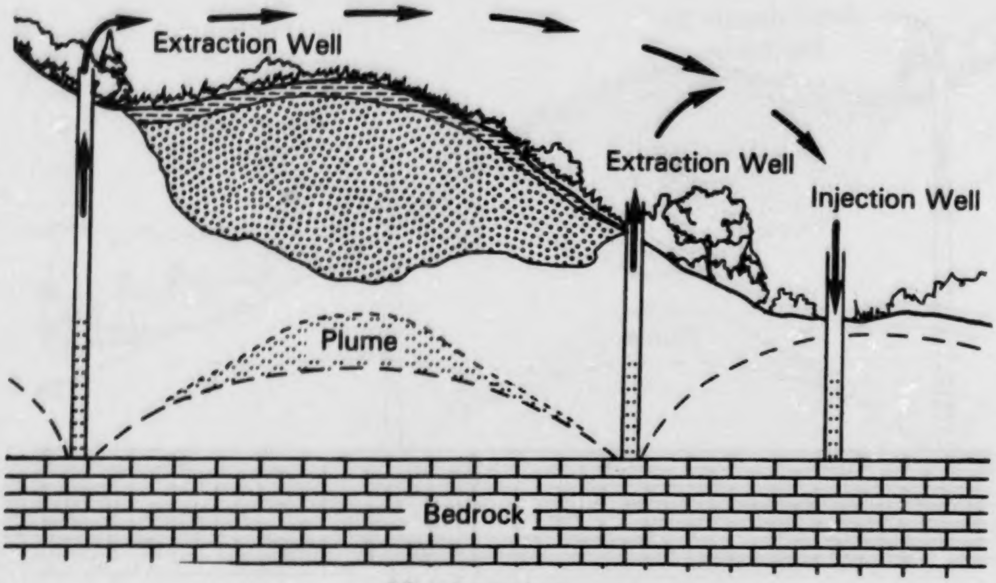
Figures 4-12, 4-13, and 4-14 illustrate these systems.

FIGURE 4-12

USE OF EXTRACTION/INJECTION WELLS FOR PLUME CONTAINMENT


Water Table
Plume
Groundwater Flow
Bedrock

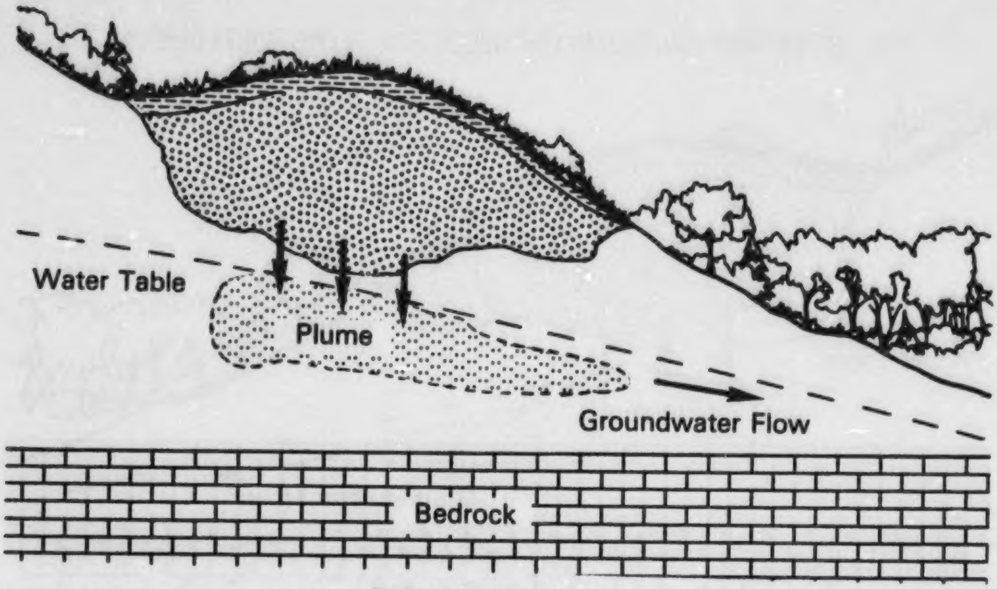
Before Pumping


Extraction Well
Extraction Well
Injection Well
Plume
Bedrock

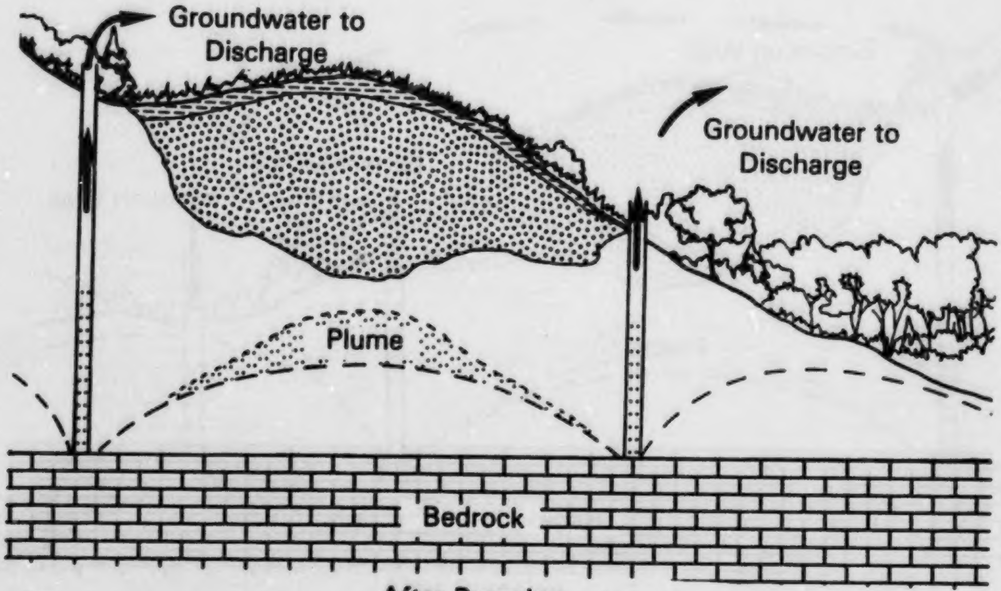
After Pumping

FIGURE 4-13

GROUNDWATER PUMPING TO CONTAIN A PLUME (NO RECHARGE)


Water Table
Plume
Groundwater Flow
Bedrock

Before Pumping

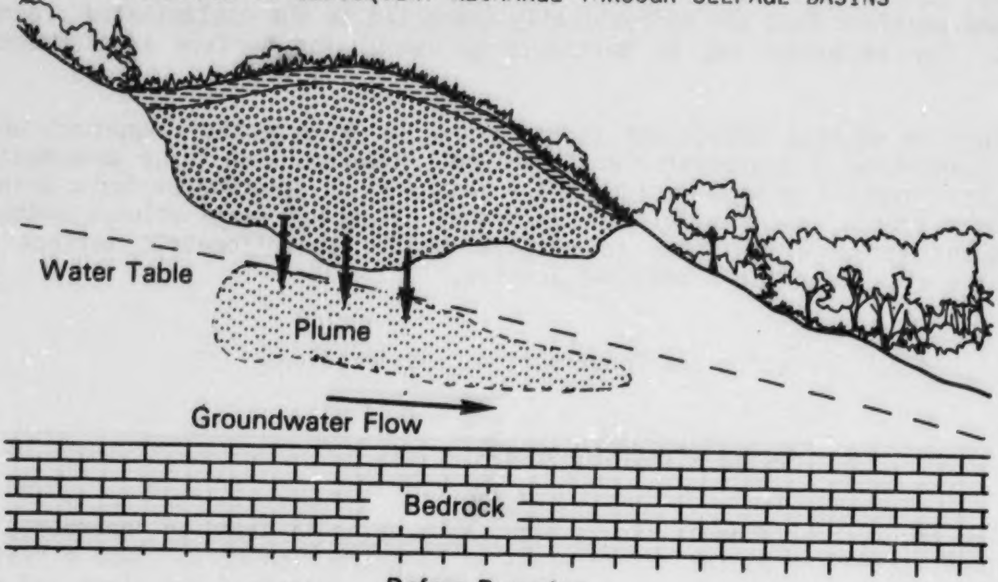

Groundwater to Discharge
Groundwater to Discharge
Plume
Bedrock

After Pumping

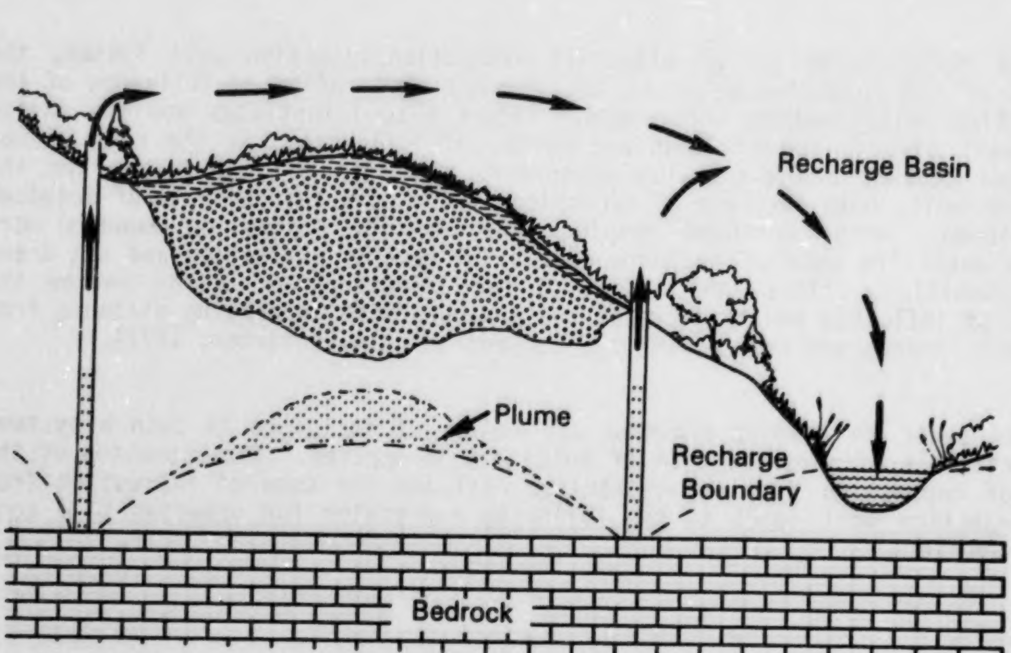
FIGURE 4-14

USE OF EXTRACTION WELLS FOR PLUME CONTAINMENT

FOLLOWED BY SUBSEQUENT RECHARGE THROUGH SEEPAGE BASINS


Water Table
Plume
Groundwater Flow
Bedrock

Before Pumping


Recharge Basin
Plume
Recharge
Boundary
Bedrock

After Pumping

4.3.2.2 Applications

Plume containment by pumping is an effective means of preventing the eventual contamination of drinking water wells or the pollution of streams or confined aquifers that are hydraulically connected to the contaminated groundwater. The technique may be particularly useful for surface impoundments.

Pumping without subsequent recharge may be an acceptable approach when small quantities of groundwater are involved. However, when large groundwater flows are involved or when residents are dependent on groundwater for a drinking water source, recharge will be necessary. Pumping large volumes without subsequent recharge may lead to changes in the potentiometric surface or direction of flow within a confined aquifer.

4.3.2.3 Design and Construction Considerations

- Extraction/Injection Wells

The theory behind containing a plume by pumping is based on incorporating the plume within the radius of influence of an extraction well. Such a system would require careful monitoring to determine the extent of the plume and any changes that may occur in the plume as pumping continues.

In order to design an effective extraction/injection well system, the effect of the injection wells on the drawdown and radius of influence of the extraction wells must be understood. Figure 4-15 illustrates how the injection well affects the drawdown and radius of influence. As the cone of depression expands and eventually encounters the cone of impression from the recharge well, both the rate of expansion of the cone and the rate of drawdown are slowed. With continued pumping, the cone of depression expands more slowly until the rate of recharge equals the rate of extraction and the drawdown stabilizes. Thus, the effect of the injection well is to narrow the radius of influence and to decrease the drawdown with increasing distance from the well (Freeze and Cherry, 1979; U.S. Department of Interior, 1977).

In order to predict drawdown and radius of influence in such a system, the principle of superposition of solutions is applied. The summation of the cone of depression from the extraction well and the cone of impression from the injection well leads to the following expression for drawdown in a confined aquifer:

$$s = \frac{Q}{4\pi T} \quad W(u) - W(u_i)$$

$$\text{where:} \quad u = \frac{r_e{}^2 S}{4Tt}$$

$$u_i = \frac{r_i{}^2 S}{4Tt}$$

The distance between the discharge well and a piezometer is called r_e and the distance between the recharge well and the piezometer is called r_i (Freeze and Cherry, 1979). All other parameters are the same as defined for the Theis equation in Section 4.3.

For a groundwater plume that is contained in the original radius of influence of the extraction well, the injection well would have the following effects:

- It may reduce the radius of the influence of the extraction well to the point where the entire plume may not be contained within the radius of influence.
- The increased pressure from the injection system will tend to move the plume towards the extraction well, thereby partly negating the effect of the decreased radius of influence.

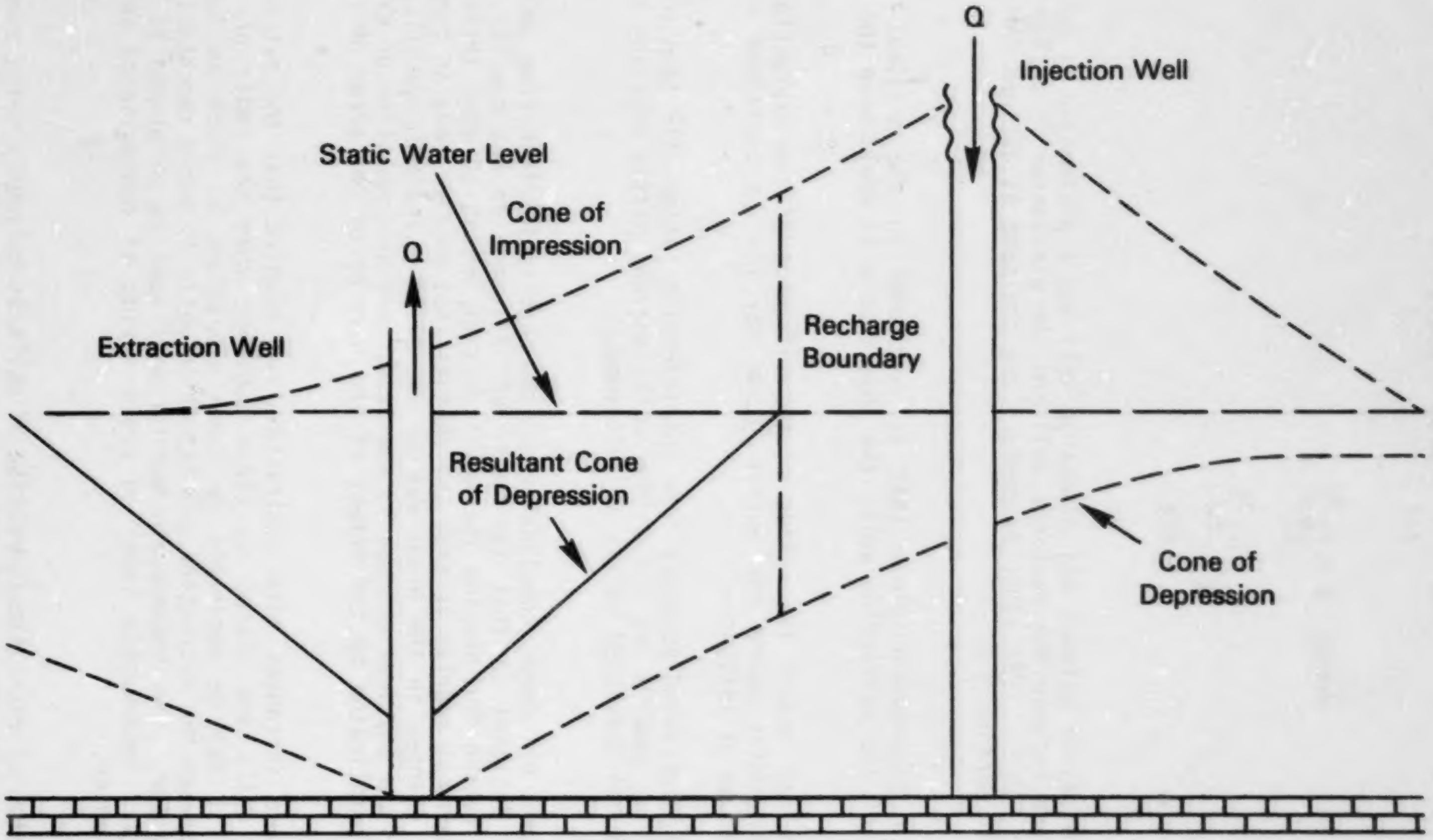
Because of these complications, the extraction/injection well system should be designed so that the radii of influence do not overlap. Another important reason for placing the wells distant enough so that their radii of influence do not overlap is that any changes that must be made in pumping as a result of changes in the plume due to age of the landfill, quantity of precipitation and physical changes in the size, such as compaction or excavation, would be complicated by the effect of the overlap of the areas of influence.

In some instances site limitations may require that the extraction and injection wells are placed so close together that the radii of influence overlap. It may be desirable in these situations to place an impermeable barrier between the extraction and injection wells to avoid recontamination of recharge water. The impermeable barrier will need to be placed to the depth of the first impermeable layer to avoid mixing of contaminated and noncontaminated water.

A system of extraction/injection of wells is currently being used in Palo Alto, California, to prevent salt water intrusion. The system designed for

FIGURE 4-15

EFFECT OF AN INJECTION WELL ON THE CONE OF DEPRESSION


Q
Injection Well
Static Water Level
Cone of Impression
Q
Extraction Well
Recharge Boundary
Resultant Cone of Depression
Cone of Depression

Palo Alto used a series of nine well pairs, with a total pumping capacity of 2.0 MGD. This corresponds to an average pumping rate of 150 gpm. Pumping is conducted in an aquifer 45 feet deep, with a transmissivity of 8,700 gpd/ft and a coefficient of storativity equal to 3.6 x 10^{-5}. Optimum spacing between well pairs was determined to be 1,000 feet (Shealan, 1977).

- Groundwater Pumping System Without Recharge

Applying the principles of radius of influence, it is possible to contain a plume by extraction alone. There are both advantages and disadvantages to this system as compared to the extraction/injection system. The withdrawal system does not incur the added pumping and maintenance costs for an injection system, but also does not have the advantage of replenishing the groundwater supply. Use of an extraction system alone would be best suited to sites where low rates of pumping are required or where the aquifer water supply is not needed as a drinking water source.

The design of an extraction system is considerably less complicated than the previously mentioned extraction/injection well systems, since the effects of the cone of impression from the injection wells do not need to be considered in determining the radius of influence needed to contain a plume. However, effective use of this system will still depend on the accurate and frequent monitoring of the plume and on a flexible design that can be adjusted as the plume changes.

- Groundwater Pumping with Recharge Through Seepage Basins

As a less costly alternative to recharging water through injection wells, seepage basins or recharge basins can be used. Since seepage basins require a high degree of maintenance to insure that porosity is not reduced, they would not be practical where several basins are required for recharge of large volumes of water or where adequate maintenance staff is not available.

As is the case for extraction/injection well systems, the effects of recharge on the cone of depression must be accounted for in designing a system that will contain the plume. Ideally, the recharge basins should be located outside the area of influence of the extraction wells.

The dimensions of a recharge basin vary considerably. The basin should be designed to include an emergency overflow and a sediment trap for runoff from rainwater. The side walls of the basin should be pervious since considerable recharge can occur through the walls (Tourbier and Westmacott, 1974).

- Well Systems for Plume Containment

Well point systems will be suitable for extracting a plume in some cases; these systems are more flexible and can be readily adjusted to account for changes in the plume. Well point systems are discussed in Section 4.3.1. However, in most instances, high capacity wells will be needed.

In choosing the size of the well casing, the controlling factor will be the size of the pump expected to be required for the yield of the well needed to incorporate the plume. The diameter of the well casing should be larger than the nominal diameter of the pump. Generally, a 4-inch submersible pump will be used for extraction volumes of less than 100gpm, and a 6-inch pump for yield requirements of 150-400 gpm. A low pumping rate will be required in many instances, and a jet ejector pump may be used.

The number of pumps needed to contain the plume will be determined by applying the equations for composite radius of influence, or for superposition of solutions if the radii of influence of the extraction and injection systems overlap.

In many instances it will be desirable to pack the wells with gravel. An artificially gravel-packed well usually costs more to construct, but is favored for such geologic conditions as:

- Fine uniform soils where use of gravel packing would allow larger slot openings in the well screen
- Thick artesian aquifers so that the entire aquifer could be screened
- Loosely cemented sandstone where small-slot well screens would otherwise be required
- Thinly bedded formations where it would otherwise be very difficult to choose the length of screen and slot sizes without knowing the thickness and nature of each stratum (Johnson Division, 1975).

The optimum length of the well screen is chosen with relation to the thickness of the plume, and the required drawdown. It will be necessary to install the well screen or wells to the first impermeable layer in order to contain the plume.

An example of an effective system for plume containment is currently operating at the Rocky Mountain Arsenal, and this system is undergoing design for expansion. Groundwater is extracted, treated, and recharged through injection wells to the other side of an impermeable barrier. The completed system will handle a flow of 443 gpm and will extend for 5,200 feet. The system will consist of about 33 extraction wells, most of which are 8 inches in diameter, and approximately 40 injection wells with a diameter of 16 to 18 inches. The extraction and injection systems are separated by an impermeable barrier to prevent mixing of contaminated and uncontaminated water. (U.S. Army Toxic and Hazardous Materials Agency, 1979.)

4.3.2.4 Advantages and Disadvantages

The advantages and disadvantages of the systems proposed for plume containment are listed in Table 4-15.

TABLE 4-15

ADVANTAGES AND DISADVANTAGES OF GROUNDWATER PUMPING TO CONTAIN A PLUME

Advantages	Disadvantages
● System may be less costly than construction of an impermeable barrier	● Plume volume and charactertics will vary with time, climatic conditions, and changes in the site. This will result in costly and frequently monitoring
● High degree of design flexibility	● System failures could lead to contamination of drinking water
● Moderate to high operational flexibility, which will allow the system to meet increased or decreased pumping demands	● O & M costs are higher than for artificial barriers

4.3.2.5 Cost for Well Construction and Pumping

Table 4-16 lists unit costs for well construction and installation suitable for containing groundwater plumes. Well point systems may also be used, and costs for these have been presented in Section 4.3.1.5.

TABLE 4-16

COSTS FOR WELL CONSTRUCTION AND INSTALLATION

Unit	Costs	Source
Wells; construction and installation without casing	\$2-2.5 per inch diameter per foot of depth	1
Casing		
4 inch PVC	\$4.50/ft	2
6 inch PVC	\$6.50/ft	
8 inch PVC	\$10.50/ft	
4 inch Submersible Pump-180 feet; 23gpm	\$1,175	3
Recharge basins		
Excavating costs using a backhoe	\$1.5/yd^3	3
Hauling; assume one mile round trip	\$1.8/yd^3	3
Retaining wall using stone filled gabions	\$76/linear foot	3
Sand liner (including transportation costs)	\$7/yd^3	3

---

[1] U.S. Department of Interior, 1977
[2] Leazer Pumps and Wells, 1980
[3] Godfrey, 1980

The following example illustrates the actual costs for an extraction/injection well system:

> A plume has contaminated an aquifer to a depth of 35 feet. The dimensions of the plume are 2,000 feet by 750 feet by 35 feet. In order to contain the plume, a series of extraction wells will be installed. The contaminated waste will also be treated (see Appendix B and Section 4.3.3), and recharge to the aquifer through a series of injection wells. The extraction system will consist of seven wells which will be pumped to a depth of 35 feet with 4-inch submersible pumps. The injection well system will be similarly designed, except that there will be four additional backup wells in case of clogging. In addition to the extraction/

injection wells, four monitoring wells will be installed to monitor the plume.

A network of 8-inch diameter pipe will carry the water from the extraction system to the treatment system, and finally to the injection wells. The distance between the extraction wells and injection wells will be 1,000 feet to avoid overlap of the radius of influence of the extraction and injection systems. Four thousand feet of piping will also be required to carry the water to and from the individual wells. Detailed costs are shown below.

- 22 wells for extraction/injection & monitoring
  -- ($2.5/inch diameter; foot) (6 inches) (35 feet) (22 wells) = $11,550
  -- ($6.5/foot casing) (35 feet) (22 wells) = 5,000
- 4-inch submersible pumps for each well = 25,850
  -- ($1,175/pump) (22 pumps)
- 8-inch steel piping = 230,000
  -- ($46/linear foot) (5,000 feet)

Total = $272,400

4.3.3 Contaminated Water Treatment

4.3.3.1 General Description

Groundwater pumping systems can be coupled with treatment systems designed for specific groundwater contamination problems. There is considerable flexibility in the design of treatment systems. The system described in this section and illustrated in Figure 4-16 is based on a modular treatment unit designed by TACION for industrial wastewater treatment and is intended to remove organics and inorganics at tertiary treatment efficiences (TACION, Inc., 1979). Appendix 3 discusses wastewater treatment modules in some detail, and the reader is referred to that section for additional information on available treatment options.

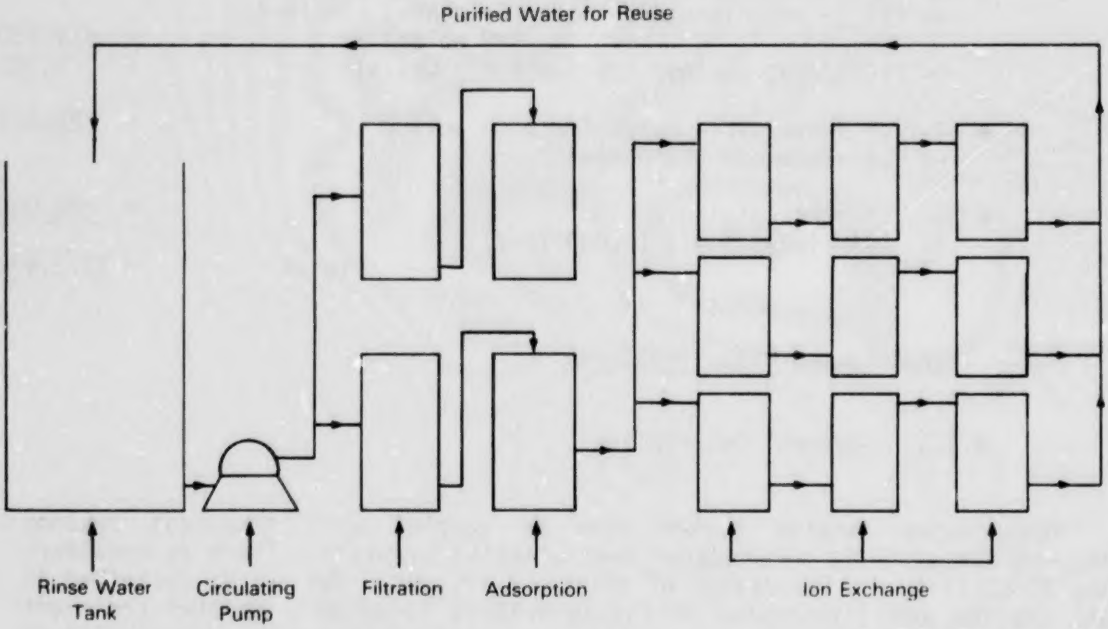
4.3.3.2 Applications

Groundwater treatment systems should be used in conjunction with pumping to lower a water table or to contain a contaminated plume. In the case of lowering the water table, it may be necessary to treat the contaminated water before disposing of it. Pumping to contain a plume will require accompanying groundwater treatment if the water is to be recharged to the aquifer or released to surface discharge. Groundwater treatment systems can renew drinking water supplies and prevent contamination of streams or aquifers that are hydraulically connected.

FIGURE 4-16

FLOW SCHEME OF TACION APPARATUS FOR INDUSTRIAL APPLICATIONS

(Source: TACION, 1979)


Purified Water for Reuse
Rinse Water Tank
Circulating Pump
Filtration
Adsorption
Ion Exchange

4.3.3.3 Design and Construction Considerations

A pipeline network from the discharge wells is needed to deliver the effluent to the rinse water tank. From there it should be pumped at a controlled rate to a dual-media filter for removal of suspended solids. In dual-media filters used for tertiary treatment, a combination of sand and graded coal are often used. The usual rate of water application is about 6 gpm/ft^2.

Dual-media filters can tolerate suspended solid loads of about 120 mg/l for filter runs of 1 to 3 days at 6 gpm/ft^3 before backwashing is required. At this loading, then, backwash will be needed every 1 to 3 days (Linsley and Franzini, 1979).

After removal of suspended solids, water is pumped under pressure to carbon adsorption units. Carbon is very effective in treating certain classes of organics. Activated carbon will effectively remove pesticides and PCB's, and in general its treatment capabilities can be summarized as follows:

- Molecules low in polarity and solubility tend to be preferentially adsorbed. Polar groups with a high affinity for water usually diminish adsorption from aqueous solutions.
- Strongly ionized materials are poorly adsorbed.
- Unless the carbon pores become physically blocked, large molecules will adsorb more easily than small molecules (Ford, 1977).

Adsorbate removal kinetics can be conducted in batch assays to determine the effectiveness of carbon for specific organics and mixtures of organics.

General criteria used for carbon adsorption where no biological treatment is included are (Culp, 1978):

Contact time	10 - 50 minutes
Hydraulic loadings	2 - 10 gpm/ft
Backwash rate	15 - 20 gpm/ft^2
Load	500 - 1,800 lb carbon/10^6 gal

The final step in the treatment system is a series of ion exchange units for removal of inorganics. The design for the ion exchange system depends on the nature of the inorganics present, since it is possible to use several combinations of various cation and anion exchange resins. The least selective method for removing toxic metals is to use a column of strong acid cation exchange in either the acid or sodium cycles. Another method involves the ability of certain weakly basic resins to form complexes with the transition metals and with the metals immediately following the transition element (Culp, 1978). Chelating resins are selective for removing metal capable of forming coordinating complexes. Copper, nickel, lead, and zinc can readily be removed with chelating resins (Dow Chemical, 1976).

In many applications, a toxic ion will be present in small amounts together with large amounts of a relatively innocuous ion of the same or higher valence. Specific ion exchangers have been developed for removal of specific ions in solution, and these should be considered in order to avoid needlessly high regeneration costs.

The costs of regeneration of carbon and ion exchange resins on-site may be prohibitively high, especially when the site is remote. It may be desirable to use a system with replacement modules so that the carbon and resins can be regenerated off-site. TACION has such a system on the market; the system uses replacement and returnable modules, and no in-plant regeneration is necessary. The system consists of filtration, carbon adsorption, and ion exchange, and can be tailored for specific wastewater problems.

4.3.3.4 Advantages and Disadvantages

The advantages and disadvantages of physical-chemical treatment of contaminated groundwater are listed in Table 4-17.

4.3.3.5 Costs

Table 4-18 lists costs for the modular treatment system for a range of volumes of groundwater. Costs for individual treatment modules are presented in Appendix B.

4.4 INTERCEPTOR TRENCHES

The use of interceptor trenches or ditches can be very effective, depending on the geohydrology of the site, in lowering the local water table and in controlling the direction of groundwater flow at a site. The design and application of interceptor trenches is discussed in detail in Section 5.2, Drainage Ditches, and therefore is not discussed further here.

TABLE 4-17

ADVANTAGES AND DISADVANTAGES OF THE GROUNDWATER TREATMENT SYSTEM

Advantages

- Achieves tertiary treatment of groundwater
- High design flexibility
- Permits water to be pumped back into aquifer with little change in potentiometric surface
- Good reliability

Disadvantages

- Large quantities of costly adsorption and ion exchange material are required
- Costs for regeneration of carbon and resins are high
- Potential for clogging of adsorption and ion exchange material

TABLE 4-18

COSTS FOR PHYSICAL--CHEMICAL TREATMENT SYSTEMS[1]

Treatment	Costs	Reference
"TACION" Modular Treatment System w/filtration, carbon adsorption and ion exchange.		
0.04 MGD	$5.3995 for equipment and material $1,100 for 3 months regeneration	TACION, 1979
0.06 MGD	$84,995 for equipment and material; $2,975 for 3 months regeneration	TACION, 1979
0.18 MGD	$225,995 for equipment and materials; $9,380 for 3 months regeneration.	TACION, 1979

---

[1] Pumping costs not included.

4.5 BIORECLAMATION

4.5.1 General Description

As previously described, in cases where a disposal site is located near a water table, groundwater flow can become contaminated with hazardous materials due to the leaching of toxic substances from the refuse site. Contaminated groundwater can be controlled by installing impermeable barriers or permeable treatment beds, by groundwater pumping, or by a bioreclamation process, if the contaminants are biodegradable. The last method is discussed in this section.

Bioreclamation is an in-place groundwater treatment technique based on the concept of utilizing microbial organisms combined with aeration and addition of nutrients to accelerate the biodegradation rate of the groundwater contaminants. Bioreclamation is a newly developed technique that has great potential for controlling contaminated groundwater flow.

4.5.2 Applications

Bioreclamation has been previously demonstrated to be an effective method of controlling groundwater contamination from underground hydrocarbon spills. The method may also be applied to a clean-up operation of groundwater contaminated by organic hazardous waste from landfills.

According to Suntech, Inc., a pioneering firm in the field of bioreclamation, the technique can be effectively used to clean up underground hydrocarbon plumes that contaminate the groundwater. However, certain organic substances, such as chlorinated solvents, cannot be very effectively controlled (A.M. Kirby, 1980).

4.5.3 Design and Construction Considerations

Microbiological research has identified several species of microorganisms that have the capability of degrading hydrocarbons and certain organic compounds to carbon dioxide, water, and other basic molecules. The microorganisms that degrade hydrocarbons are tentatively identified as bacteria belonging to the genera Pseudomonas and Arthrobacter (Raymond et al., 1976). Results from a pilot study to determine the growth rate of indigenous microbial flora have shown that the small, natural microbial population could be increased a thousandfold by adding air, inorganic nitrogen, and phosphate salts to the groundwater (Jamison et al., 1975). Utilization of the hydrocarbon substances by microbial activities was also found to be proportional to the growth rate of the microbial population.

Application of the bioreclamation method to control of contaminated groundwater from waste disposal sites may require slight modifications to the currently employed method. Groundwater contaminated with materials that leached from a disposal site may contain a great variety of hazardous substances besides hydrocarbon compounds. Therefore, when the bioreclamation technique that was originally developed for the "in-situ" clean-up of groundwater contaminated with hydrocarbons is used, it is necessary to adjust certain factors of the process to accommodate the removal of contaminants that may comprise a wide range of toxic materials.

It is recommended that the contaminated groundwater be studied to determine the chemical constitutents to be removed. Once the contaminants are identified, appropriate bacteria can be chosen to accomplish the desired degradation process.

The general method of treating contaminated groundwater with the bioreclamation method is illustrated in Figure 4-17. First, wells are placed at strategic locations with respect to the contaminant plume. Then the chosen microorganisms are injected into the groundwater along with oxygen and nutrients. Prior to the injection, the bacteria should be acclimated to the wastes they are intended to treat. To promote microbial action, a proper balance of oxygen and nutrients is maintained by continuous pumping, makeup, and reinjection into the groundwater.

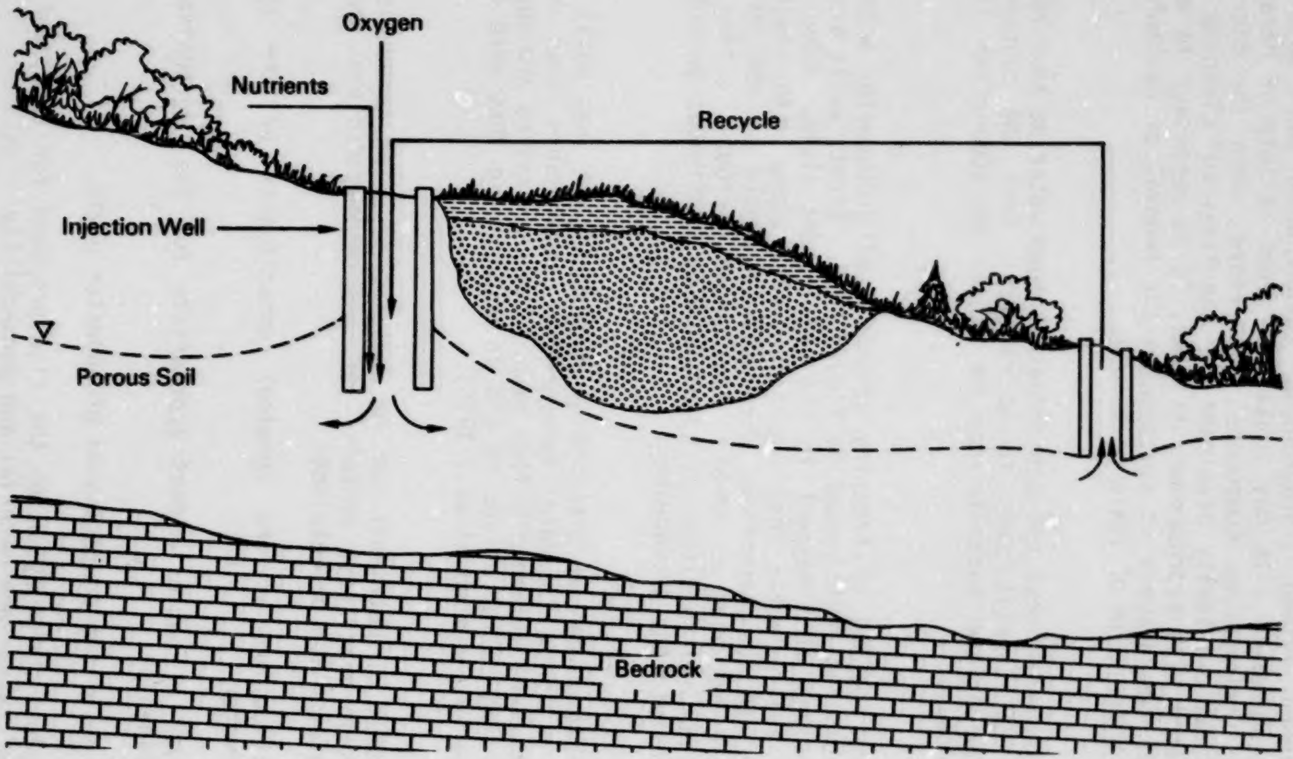
Proper aeration can be obtained by purging oxygen into wells by the use of diffusers attached to paint-sprayer-type compressors that can deliver oxygen at a constant volumetric flow rate. The compressors are equipped with pressure gages and relief valves to aid in determining that each diffuser is operating properly (Raymond et al., 1976).

The addition of nutrients can be achieved by using conventional pumps. For actual design practices, Suntech, Inc. has recommended that the following preliminary information be obtained:

- Identification of the chemical constituents of the contaminated groundwater
- The type of bacteria most appropriate for the degradation of these contaminants
- The size of the contaminated groundwater plume
- Geological information on the site proposed for treatment, including type of subsurface material and permeability
- The volumetric flow rate of the groundwater flow and the level of contamination.

FIGURE 4-17

TREATMENT OF THE CONTAMINATED GROUNDWATER WITH THE BIORECLAMATION TECHNIQUE


Oxygen
Nutrients
Recycle
Injection Well
∇
Porous Soil
Bedrock

After the treatment process has started, is is advisable to monitor the treated groundwater flow to ensure the adequacy of the treatment system.

4.5.4 Advantages and Disadvantages

The advantages and disadvantages of the bioreclamation method are summarized in Table 4-19.

TABLE 4-19

ADVANTAGES AND DISADVANTAGES OF THE BIORECLAMATION TECHNIQUE

Advantages	Disadvantages
● Good for removal of hydrocarbons and a limited amount of organic material from contaminated groundwater	● Does not remove chlorinated solvents or heavy metals
● Environmentally sound	● Introduction of nutrients containing phosphate and nitrogen may have adverse effects on the surface water stream located near the treatment site
● Fast, safe and economical	● Excessive breakdown of equipment such as pumps, compressors, and diffusers may occur, resulting in higher maintenance and operational cost
● Inexpensive materials used	● Long-term effectiveness of this method is unknown
● Good for short-term treatment of contaminated groundwater	

4.5.5 Cost

It is difficult to present a detailed estimate of the cost of treating contaminated groundwater using the bioreclamation method, since it involves a large number of variables. According to A.M. Kirby of Suntech, the cost of treating a 5-acre area contaminated with petroleum crude would be approximately $50,000 based on a 6-month clean-up period. But this cost is only applied to a clean-up of petroleum waste. Treatment cost for groundwater contaminated with other hazardous waste materials could run up to several million dollars, depending upon the size of the contaminated groundwater plume, the level of contamination, the geology of the soil, and the length of time needed for complete removal.

REFERENCES

ARMCO, Inc., Baltimore, MD. April 1980. Personal communication with P. Le.

AFTES (Association Francaise des Travaux en Souterrain). 1975. Recommendations for the use of grouting in underground construction, trans. G. W. Clough.

Avanti International. 1979. AV-100 chemical grout safe operating practices program. Houston, TX.

Baver, L.D., W.H. Gardner, and W.R. Gardner. 1972. Soil physics. New York: John Wiley & Sons, Inc.

Bowen, R. N. C. 1975. Grouting in engineering practice. New York: Halsted Press.

Boyes, R. G. H. 1975. Structural and cut-off diaphragm walls. London: Applied Science Publishers, Ltd.

Calgon Company, Pittsburgh, PA. 1980. Personal communication with P. Rogoshewski.

Culp, R. L. et al. 1978. Handbook of advanced wastewater treatment. New York: Van Nostrand Reinhold Environmental Engineering Series.

D' Appolonia, D. J. 1980. Soil bentonite slurry trench cut-offs. Journal of the Geotechnical Engineering Div., ASCE 106(4):399-717.

Davis, T., Johnson Divison, UOP. April 1980. Personal communication with K. Wagner.

Dow Chemical. 1977. A basic reference on ion exchange. Form No. 1976-194-77.

Fair, G. M., J.C. Geyer, and D.A. Okun. 1966. Water and wastewater engineering, vol. 1. New York: John Wiley & Sons, Inc.

Ford, D. L. 1977. Putting activated carbon in perspective. Reprint Engineering Science, Inc., Austin, TX.

Freeze, R. A., and J. A. Cherry. 1979. Groundwater. Englewood Cliffs, NJ: Prentice-Hall, Inc.

Frey Co., W. S., Virginia. April 1980. Personal communication to P. Le.

Germany Valley Limestone Co., WV. April 1980. Personal communication to P. Le.

Godfrey, R. (ed.). 1979. Building construction cost data, 1980. Kingston, MA: Robert Snow Means Co., Inc.

Hantush, M.S. 1960. Modification of the theory of leaky aquifers. Journal of Geophysics Research 65(11):3713-3725.

Jacuzzi Pumps, Standard Supply, Inc., Gaithersburg, MD. April 1980. Personal communication with K. Wagner.

Jamison, V. W., R.L. Raymond, and J.O. Hudson, Jr. 1975. Biodegradation of high-octane gasoline in groundwater. Development in Industrial Microbiology, vol. 16. Washington, D.C.: American Institute of Biological Sciences, pp. 305-312.

Johnson Division, UOP, Inc. 1975. Groundwater and wells. Edward F. Johnson, Inc., Saint Paul, Minnesota.

Kirby, A. M., Suntech, Inc., April 1980. Personal communication to P. Le.

Kirk-Othmer, 1979.

Leazer Pumps and Wells, Remington, VA. April 1980. Personal communication with K. Wagner.

Linsley, R., and J. Franzini. 1979. Water resources engineering, 3d ed. New York: McGraw-Hill Book Co.

McDonough Bros. Inc., Texas; personal communication to P. Le, April 4, 1980.

McMahon, L., and P. Pereira. 1979. 1980 Dodge guide to public works and heavy construction costs. New York: McGraw-Hill Information Systems Co.

Milam, 1980.

Neuman, S. P. 1972. Theory of flow in unconfined aquifer considering delayed response to the water table. Water Resources Research 8:1030-45.

Neuman, S. P. 1973. Supplementary comments on theory of flow in unconfined aquifers considering delayed response to the water table. Water Resources Research 9:1102-03.

Neuman, S. P. 1975. Analysis of pumping test data from aniostrophic unconfined aquifers considering delayed gravity response. Water Resources Research 11(2):329-345.

Neuman, S. P., and P. A. Witherspoon. 1969a. Theory of flow in a confined two-aquifer system. Water Resources Research 5:803-816.

Neuman, S. P., and P. A. Witherspoon. 1969b. Applicability of current theories of flow in leaky aquifer. Water Resources Research 5:8817-29.

Raymond, R. L., V.W. Jamison, J.O. Hudson, Jr. 1976. Beneficial stimulation of bacterial activity in groundwaters containing petroleum products. AICh Symposium series, vol. 73, no. 166, pp. 390-404.

Shallard, S. G., Engineered Construction International, Inc., Pittsburgh, PA. 1980. Personal communication with P. Spooner.

Shealan, N. T. 1977. Injection/Extraction well system - A unique seawater intrusion barrier. Groundwater 15: 1.

Spoljaric, N., Delaware Geological Survey, Newark, Delaware. June 1980. Personal communication with P. Rogoshewski.

Spoljaric, N, and W. Crawford. 1979. Removal of contaminants from landfill leachates by filtration through glauconitic greensands. Environmental Geology 2(6):359-363.

Staples, G., JRB Associates, Inc., McLean, VA. April 1980. Personal communication with P. Le.

TACION. 1979. Tacion systems for industrial pollution control. Tacion water purification system product literature.

Tallard, G.R., and G. Caron. 1977. Chemical grouts for soils, vol. 1: Available Materials. Federal Highway Administration Report, FHWA-RD-77-50.

Tourbier, J., and R. Westmacott. 1974. Water resources protection measures in land development. U.S. Department of Interior. Office of Water Resources Research.

Universal Linings, Inc., Philadelphia, PA. 1980. Personal communication between D. Small and P. Rogoshewski.

Ueguhardt, L. C. et al. 1962. Civil engineering handbook. New York: McGraw-Hill Book Co.

U.S. Army Toxic and Hazardous Materials Agency, 1979. Environmental impact statement on groundwater treatment system for the Rocky Mountain arsenal. Draft Report.

U.S. Department of Interior. 1977. Groundwater manual - a water resources technical publication, Washington, D.C.: U.S. Government Printing Office.

U.S. Environmental Protection Agency. 1978. Guidance manual for minimizing pollution from waste disposal sites. Cincinnati, OH. EPA-600/2-78-142.

U.S. Environmental Protection Agency. 1978. Proceedings of the fourth annual research symposium held at San Antonio, TX, March 6-8. pp. 282-298. EPA-600/9-78-016.

Wenzel, L. K. 1942. Methods of determining permeability of water-bearing materials. U.S. Geol. Survey Water Supply Paper 887.

5.0 LEACHATE COLLECTION AND TREATMENT

Leachate is defined as the contaminated liquid discharged from a waste disposal site to either surface or subsurface receptors. Water percolating through the surface of a landfill eventually saturates the waste to its field capacity moisture content. At that stage, moisture may percolate to the ground below or seep from the side of the landfill. A leachate collection system is designed to intercept the leachate before it becomes a problem by contaminating groundwater or surface waters. The water is subsequently treated or discharged depending on the extent of contamination.

Leachate collection systems consist of a series of drains that intercept the leachate and channel it to a sump, wetwell, or appropriate surface discharge point. The drains may consist of open ditches or trenches that may or may not include pipes or tile drains. The sump, wetwell, or other collection basin is also part of the collection system and from here the water is pumped to treatment or an appropriate discharge point. Leachate treatment will be highly variable depending on the composition and strength of the leachate.

Liners may also serve as a passive interceptor for leachate. Although liners have widespread application for leachate collection in new landfill sites, technical and economic factors markedly restrict their use for existing sites.

Leachate collection systems are applicable to control of surface seeps and seepage of leachate to groundwater. Leachate seeps on slopes are caused when surface water infiltrates the cover soil, migrates downward until it encounters a less permeable intermediate soil layer or refuse layer, and then moves laterally until it seeps through the soil cover. Groundwater contamination by leachate results from water percolating through the site as well as from lateral groundwater inflow through the fill material.

In many cases, disposal sites are placed in low-lying areas, mined-out areas, or valleys. All these situations are natural pathways for drainage flows or streams. These streams must be protected from contamination and from entering the disposal site and increasing leachate generation. It may be necessary to divert these natural flows or to collect them in separate drainage systems.

Prior to system design, detailed studies must be undertaken. These should include delineations of the site's topographic and geographic setting, waste characterization, leachate characterization, and leachate treatability studies.

5.1 SUBSURFACE DRAINS

5.1.1 General Description

Subsurface drains consist of underground, gravel-filled trenches generally lined with tile or perforated pipe. The drains intercept leachate or infiltrating water that is destined to become leachate and transport it away from the site.

5.1.2 Applications

Subsurface drains can be used to intercept leachate or infiltrating water in any clay or silty clay soil where the permeability is not adequate to maintain sufficient flow and at sites where the leachate is not too viscous or gummy to prevent flow to the drains. Other conditions, such as a deep frost zone, may also restrict the use of underdrains in certain soils. Subsurface leachate collection systems have been proposed or constructed at several existing landfills. Examples include Love Canal (Glaubinger et al., 1979), and the Rossman's Landfill in Oregon City (Solid Waste Management, 1979). The layout for the collection system at Love Canal and at the Rossman's Landfill is considered in more detail in Section 5.1.3.

5.1.3 Design and Construction Consideration

The objective of this section is to provide the user with both theoretical and practical aspects for the design and construction of leachate collection systems.

Design of a suitable leachate collection system requires that the quantity of leachate produced can be reasonably estimated. The water balance equation is generally used as a basis for predicting leachate production and is generally expressed in inches per month or per year (EPA, 1979).

$$\text{Percolation} = P - R - AET - S$$

where:
P = precipitation (inches)
R = runoff (inches)
AET = actual evapotranspiration (inches)
S = gain in moisture storage within the soil (inches)

Moisture storage is a function of pore characteristics and thickness of the layer and is determined by obtaining the best estimate of available water content, based largely on soil type, and then adjusting the estimate to account for the thickness of the root zone (EPA, 1979).

The water balance equation can be conducted on several levels of detail depending upon the need. It can be integrated for any desired length of time and over any chosen number of time steps. The necessary tables and procedures for estimating the water balance can be obtained from a paper by Thornthwaite and Mather (1957).

The quantity of leachate produced, as predicted by the water balance equation, can then be used in estimating drainage requirements, i.e. size and spacing.

Aside from precipitation, there are a number of other sources of leachate. In some situations, groundwater flow through the site may contribute significantly to the generation of leachate. This is particularly true of unlined disposal sites. This factor should be considered in estimating drainage requirements for all sites. Estimates of the volume and velocity of groundwater flow can be obtained through observations and computations discussed in Section 4.3.1.3. Other possible sources of leachate at the disposal site are the production of liquids from biodegradation of some organic compounds and the dumping of liquids at the site. These should also be considered when estimating drainage requirements.

Once the quantity of leachate has been estimated, the design of the collection system is undertaken. The collection system will consist of a sump, basin, or wetwell for final collection and a series of drains and/or wells located within, directly beneath, or in the immediate proximity of the landfill so as to maximize collection of leachate and minimize collection of clean water.

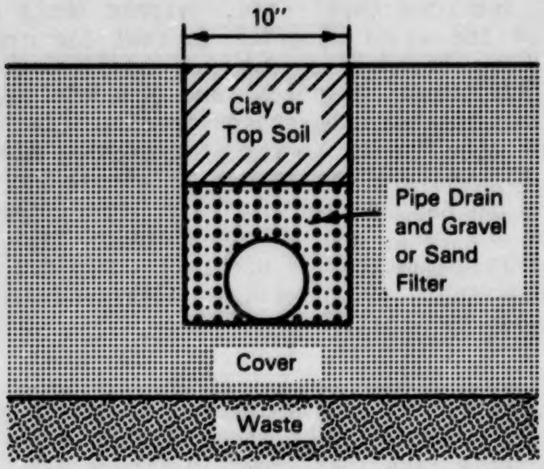
Subsurface leachate collection systems have been proposed or constructed at several existing landfills. The drainage systems are generally constructed by excavating a trench and laying tile or piping end to end in strings along the bottom. The trench is then backfilled with gravel or other envelope material to a designated thickness and then the rest of the trench is backfilled with soil. Often the gravel is lapped with fabric to prevent fine soil from entering the gravel and clogging the drain. The front-view of a subsurface leachate collection system is illustrated in Figure 5-1.

In some instances, gravel-packed wetwells may be used. Wells are constructed similarly to trenches.

FIGURE 5-1

SUBSURFACE LEACHATE COLLECTION DRAIN

(Source: EPA, 1979)



An impermeable liner may be required on the down-gradient end of the subsurface drain to prevent the flowthrough of intercepted and contaminated groundwater if the surrounding materials have a moderately high to high permeability.

The flow path of water from the ground surface to the drainage system consists of four phases: (1) flow from the ground surface through the fill and to the saturated water level; (2) flow to the sides and bottom of the trench; (3) flow through the backfill and envelop material through the tube joints of perforations; (4) flow into the tube itself (Van Schlifgaarde, 1974).

The major design problem for subsurface drains is to determine the optimum spacing, depth and hydraulic capacity. Determination of these criteria is usually based on practical experience, experimental data, and calculations using drainage formula. Spacing between drain lines and wetwells depends upon the depth of the drain below the surface, the hydraulic conductivity of the soil, the amount of subsoil to be drained, and the potential for constructing underdrains beneath the landfill.

Where depth and spacing are determined experimentally, they are usually determined from such properties as soil texture, hygroscopic moisture, and specific surface of soil particles as well as from the depth of fill material. (Van Schlifgaarde, 1974).

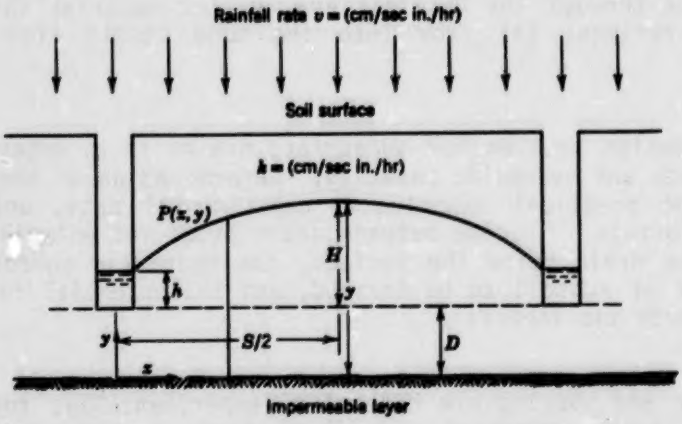
Design equations that have been developed for flow to a drainage pipe indicate that a greater depth allows for wider spacing. These formulae are considered in this section in relation to spacing. This will be an important consideration when designing a collection system for waste disposal sites; while it is necessary to intercept all leachate beneath the site, it is frequently undesirable or hazardous to excavate through the waste material. In positioning drains at the Love Canal site, seismic tests were used to establish the boundaries of the waste material so that the drains could be positioned outside of these boundaries and yet still intercept all leachate (Glaubinger et al., 1979). For practical and economic reasons, however, the depth of tube drains cannot always be chosen freely. Limiting factors include impermeable layers and operational limits of trenching machinery. It will be technically feasible to excavate the trench to almost any desired depth, but the economics of doing so may be prohibitively high. Hydraulic backhoes can excavate to depths of 55 feet, and if the depth of fill exceeds 55 feet, a crane and clam-shell arrangement can be used. The minimum depth of the drains is determined by the strength of a drain to withstand expected loads and by the risk of damage caused by frost.

The flow of groundwater to a drain pipe or ditch is governed by the same factors that control flow to a well (see Chapter 4.3). Both ditches and drains create a water table like that shown in Figure 5-2.

FIGURE 5-2

DIAGRAM OF HOOGHOUDT'S DRAIN-SPACING FORMULA

(Source: Baver et al., 1972)



The cone of depression observed around a well becomes a trough along the line of the drain (Linsley and Franzini, 1979). The spacing of the drains must be such that the water table at its highest point between drains intercepts all leachate-generating wastes, and does not interfere with plant growth or zone of aeration, if these factors play a part in proper operation of the fill.

In actual practice, spacing of underdrains may be restricted by the boundaries of the wastes in such a way that the composite cones of depression of the drains do not completely overlap and some leachate escapes the collection system. This may occur where ideal spacing requires that underdrains be constructed beneath a waste site. While horizontal boring techniques are available, the logistics of installing a gravel-filled trench beneath the site may be prohibitive. Since the drain spacing is influenced by depth and hydraulic conductivity, it may be possible to increase spacing and still intercept all leachate by increasing drain depth and by adjusting envelope thickness to increase hydraulic conductivity so that underdrains beneath the site are not necessary.

The simplest formula for estimating drain spacing assumes homogeneous soils and one-dimensional flow. Drain spacing can be estimated from Hooghoudt's formula as follows (Baver et al., 1972) (see Figure 5-2).

$$S = \frac{4k}{Q} \left[(D + H)^2 - (D + h)^2\right]$$

where S = drain spacing (feet)

k = hydraulic conductivity (feet/day)

Q = design flow to the drain (ft^3 flow/day/ft of ditch)

D = depth of flow layer beneath the drains (feet)

H = height of groundwater table above the plane through the drains and midway between two drains (feet)

h = height of water level in the drain (feet)

This equation has been widely used to calculate drain spacing from permeability, flux, and desired water level. despite the fact that actual flow to a drain is not one-dimensional. Hooghoudt's equation is based solely on Darcy's law, discussed previously in Chapter 4.3. However, since flow is seldom one-dimensional, the solution to many drainage problems is based on the Laplace equation, which combines the concepts of Darcy's Law and the equation of continuity to determine drain spacing and flow in a two-dimensional and three-dimensional system. Laplace's equation for two-dimensional flow is represented by:

$$\frac{d^2H}{dx^2} + \frac{d^2H}{dy^2} = 0$$

where x and y are direction of flow in a two-dimensional system and H equals the height of groundwater table above the plane through the drains and midway between two drains. An infinite number of solutions of this equation exist and the problem is to find the solution that matches the specific boundary conditions. A large number of solutions of the Laplace equation have been published to date. The solution to drainage problems using the Laplace equation frequently requires use of an analog or digital computer program (Freeze and Cherry, 1979). Further consideration of the Laplace equation is outside the scope of this project.

In designing tube-drain systems for capacity, the size of the drainage pipe is determined by design flow, slope and roughness (Van Schlifgaarde, 1974). Manning's formula is generally used:

$$Q = R^{2/3} \; S^{1/2} \; A/n$$

where:		
	Q = discharge	(ft^3/sec)
	R = diameter/4	(feet)
	S = slope	(ft/ft)
	A = cross sectional area	(ft^2)
	n = roughness coefficient	

The minimum recommended roughness coefficients (n) for various conduit materials are:

● Clay tile	0.011
● Concrete tile and perforate pipe	0.011
● Vitrified clay pipe	0.011
● Perforated corrugated metal pipe	0.021
● Corrugated plastic tubing	0.017

(Source: Van Schlifgaarde, 1974)

Fired clay generally has better resistance to corrosive or high strength chemical wastes than do plastic tubing or metal pipe.

The design flow will be based on the water balance equation and on the required drain spacing.

Minimum grade or slope is determined on the basis of site conditions and size of the drains. Some designers wish to specify a minimum velocity rather than a minimum grade. It is generally desirable to have a slight slope in order to obtain a velocity sufficient to clean the drain during discharge and to speed up emptying of a drain after a discharge period (Van Schlifgaarde, 1974). Minimum recommended grades for subsurface drainage are as follows:

Drain Tube Size		
cm	in	Grade, %
10.2	4	0.10
12.7	5	0.07
15.2	6	0.05

Slopes of about 0.1 percent can be obtained with present trench digging equipment accurate to within 1 cm of the prescribed depth (Van Schlifgaarde, 1974).

Drains have a relatively small area of inflow, causing an entrance resistance. Total flow resistance, including the entrance resistance, depends on:

- Hydraulic conductivity of the material surrounding the drain
- Geometric characteristics of flow
- Geometric characteristics and distribution of the inflow openings in the tube wall

It has been found that failures of tube drains are often due to the high resistance of approach of the envelope material and soil and that the type of tube is usually less critical to performance than the envelope material and soil. Application of the proper envelope material in sufficient quantities can significantly reduce the effect of resistance (Van Schlifgaarde, 1974). The most commonly used envelope materials include sand and fine gravel, and to a lesser extent straw, woodchips, and fiberglass.

Where the envelope and base material are more or less uniformly graded, the Soil Conservation Service recommends the following as a generally safe envelope stability ratio:

$$\frac{\text{particle diameter of 15\% (by wt.) of the envelope material}}{\text{particle diameter of 85\% (by wt.) of the base soil material}} < 5$$

For placement around perforated tubing, the 85 percent (by weight) overlap particle size should be no smaller than one-half the perforation diameter (Van Schlifgaarde, 1974).

Recommendations for drain envelope thickness have been made by various agencies. The Bureau of Reclamation recommends a minimum thickness of 10 cm around the pipe, and SCS recommends a minimum of 8 cm for agricultural drains. In actual practice, much thicker envelopes may be used to increase hydraulic conductivity. An 8-inch-diameter perforated pipe used for leachate collection at Love Canal is surrounded with about 2 feet of gravel (Glaubinger et al., 1979).

After the trench is backfilled with the appropriate thickness of envelope material, it may be desirable to wrap the gravel with a fabric to prevent clogging of the gravel and drains with soil. One such available material is Typlar, a strongly woven fabric that allows liquids to pass through but prevents soil from getting into the pipeline (Solid Waste Management, 1979).

The design and construction of leachate collection systems can be exemplified by the Love Canal and Rossman's Landfill collection systems. Figures 5-3 and 5-4 illustrate the leachate collection system designed for Love Canal.

FIGURE 5-3

DESIGN PLAN FOR LEACHATE COLLECTION SYSTEM AT LOVE CANAL

(Source: Glaubinger et al., 1979)
See Copyright Notice, Page 496

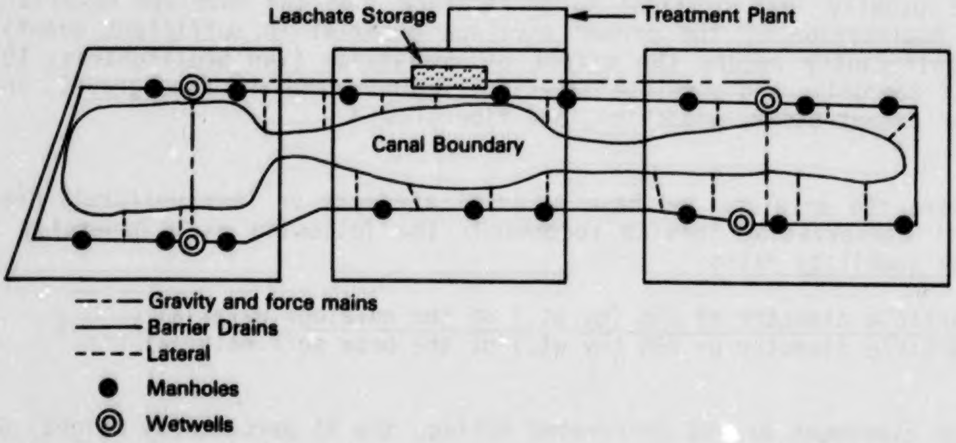
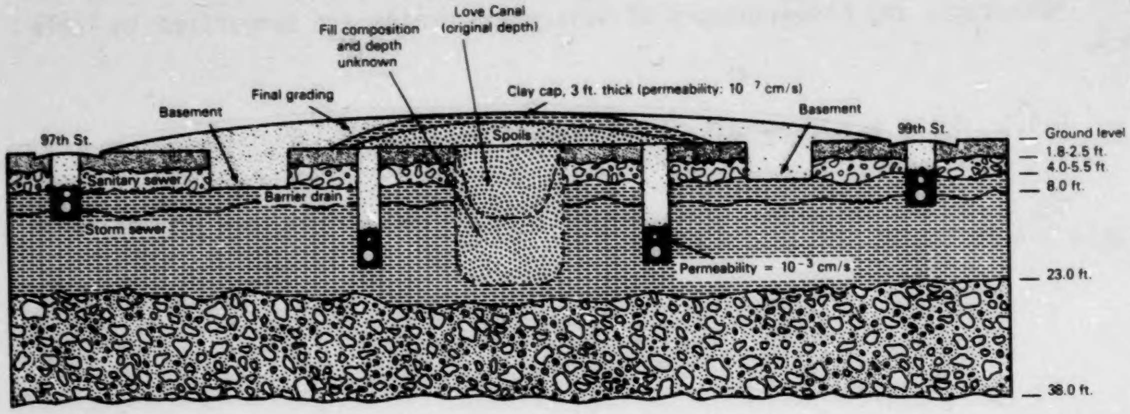

Leachate Storage
Treatment Plant
Canal Boundary
Gravity and force mains
Barrier Drains
Lateral
Manholes
Wetwells

FIGURE 5-4

LEACHATE COLLECTION SYSTEM FOR LOVE CANAL - TRAVERSE VIEW

(Source: Glaubinger et al., 1979)
See Copyright Notice, Page 496



Love Canal (original depth)
Fill composition and depth unknown
Final grading
Clay cap, 3 ft. thick (permeability: 10 $^{-7}$ cm/s)
Basement
Basement
97th St.
99th St.
Spoils
Sanitary sewer
Barrier drain
Storm sewer
Permeability = 10^{-3} cm/s
Ground level
1.8-2.5 ft.
4.0-5.5 ft.
8.0 ft.
23.0 ft.
38.0 ft.

The heart of the collection system at Love Canal is a series of drains with 6 to 8 inch-diameter perforated, vitrified clay pipe backfilled with about 2 feet of gravel envelope. The ditches run roughly parallel along the north and south borders of the canal, as shown in Figure 5-3. The trenches are approximately 12 feet below grade, dropping to a maximum of 15 feet. With a gradient of 0.5 percent, they empty leachate into precast concrete wetwells. Leachate is pumped from wetwells by vertical submersible pumps to an 8-inch-diameter gravity main, from which it descends into concrete holding tanks. Drains of different elevations are connected by manholes. To hasten dewatering from the canal, lateral trenches have also been dug between the canal boundaries and the main drainage system (Glaubinger et al., 1979).

An interesting modification of the Love Canal collection system is being designed for the Rossman's Landfill in Oregon City (Solid Waste Management, 1979). This landfill contains only municipal wastes. The system is, again, an unlined gravity-fed leachate collection system. The collection system consists of four 48-inch manholes connected to 8-inch perforated PVC pipes that run in a north-south and east-west direction to two pumps. The perforate pipes will be surrounded by about 1-1/2 inches of gravel and wrapped in a membrane of Typlar. Pumpage requirements are anticipated to be about 50 gpm, although flow may reach 100 gpm during record rainfalls. Leachate will flow by gravity to the pump station, where it will be pumped to a series of disposal fields similar to septic tank adsorption fields for treatment. Current plans are to recirculate the leachate back through the landfill (Solid Waste Management, 1979).

5.1.4 Advantages and Disadvantages

Advantages and disadvantages of subsurface drains are summarized in Table 5-1.

5.1.5 Costs for Materials and Construction

The costs for materials and construction of underdrains are presented in Table 5-4 at the end of this chapter.

TABLE 5-1

ADVANTAGES AND DISADVANTAGES OF SUBSURFACE DRAINS

Advantages	Disadvantages
● Operation costs are relatively cheap since flow to underdrains is by gravity	● Not well suited to poorly permeable soils
● Provides a means of collecting leachate without the use of impervious liners	● In most instances it will not be feasible to situate underdrains beneath the site
● Considerable flexibility is available for design of underdrains; spacing can be altered to some extent by adjusting depth or modifying envelope material	● System requires continuous and careful monitoring to assure adequate leachate collection
● Systems fairly reliable, providing there is continuous monitoring	

5.2 DRAINAGE DITCHES

5.2.1 General Description and Applications

Drainage ditches can be an integral part of a leachate collection system in that they may be used as collectors for surface water runoff, collectors leading from subsurface drains, or as interceptor drains.

Surface drainage may be essential for flat or gently rolling landfills underlain by impermeable soils where subsurface drainage may be impractical or uneconomical. Open drainage ditches have been included as part of the leachate collection system for an impoundment operated by Union Carbide (Slover, 1976). The impoundment has a clay floor and a subsurface drainage system to prevent ponding. Open ditches are included around the periphery of the site to carry off impoundment surface waters and peripheral waters that may contribute to ponding and leachate formation.

Open ditches may be used as interceptor drains to collect lateral surface seepage from the landfill, thus preventing it from percolating into groundwater or flowing laterally to an area that should be protected. The choice between using an open drain or subsurface drain depends upon the slope of the flow. For steep slope, open drains are generally more desirable.

Finally, an open ditch may be used in certain instances to intercept subsurface collectors and carry the leachate to its ultimate disposal.

5.2.2 Design and Construction Considerations

Open ditches are on the order of 6 to 12 feet deep. When they are connected to subsurface drains, they must be deep enough to intercept the underdrains (Van Schlifgaarde, 1974).

The water level in a ditch is determined by the purpose the ditch has to serve. Surface drains require sufficient freeboard when running at full capacity. The flow velocity should be kept within certain limits in view of scouring of the bed and side slopes and of sediment deposition. Important factors governing the desired flow velocity are soil type, type of channel, well roughness, and sediment load (Van Schlifgaarde, 1974). The size of ditch necessary to carry the estimated quantity of water can be determined from the Manning Velocity equation and is dependent upon the slope, depth, and shape of its cross-section.

The selection of side slopes is based on stability of soil and on the hazard of scour, taking into account possible groundwater pressures and vegetative cover. The stability of side slopes may be improved by tamping or rolling. Trapezoidal cross sections are generally most efficient. In fine-grained soils such as heavy clays, 1/2 to 1 slopes (0.5 ft to 1 ft vertical) and 1-1/2 to 1 are common. In coarser textured soils, 1 to 1 or 2 to 1 may be advisable (Van Schlifgaarde, 1974).

Ditch bottoms at junctions should be at the same elevation to avoid drops that may cause scour. Right angle junctions encourage local scour of the bank opposite the tributary ditch, and the smaller ditch should be designed to enter the larger at an angle of about 30 degrees. Scour will also occur at sharp changes in ditch alignment so long radius curves should be used where change is necessary (Linsley and Franzini, 1979).

An open ditch can be kept in efficient working condition by careful maintenance. A drain allowed to become obstructed by brush, weed growth, or sediment can no longer be efficient; it should be cleaned to its original depth when efficiency is curtailed.

5.2.3 Advantages and Disadvantages of Drainage Ditches

The advantages and disadvantages of drainage ditches are summarized in Table 5-2.

TABLE 5-2

ADVANTAGES AND DISADVANTAGES OF DRAINAGE DITCHES

Advantages	Disadvantages
● Low construction and operating cost	● Require extensive maintenance to maintain operating efficiency
● Useful for intercepting landfill side seepage and runoff	● Generally not suited for deep disposal sites or impoundments
● Useful for collecting leachate in poorly permeable soils where subsurface drains cannot be used	● May interfere with use of land
● Large welted perimeter allows for high rates of flow	● May introduce need for additional safety/security measures

5.2.4 Costs

Costs of materials and construction for open ditches are included in Table 5-4, later in this chapter.

5.3 LINERS

5.3.1. General Description and Applications

Liners for control of leachate are impermeable barriers situated beneath the landfill and that intercept the leachate before it reaches the groundwater supply. Use of subsurface liners at existing sites is limited by the logistics of placing the liner beneath the wastes. Although prefabricated liners have widespread use as bottom sealant for new sites, it is not technically feasible to use prefabricated liners for existing disposal sites. However, slurries and grouts may be injected to form a bottom seal under certain limited conditions. Possibilities for bottom seals at existing sites include:

- Formation of a bottom seal by the use of pressure-injected grouts
- Use of a bentonite slurry that would be allowed to settle to the bottom of a lagoon, thereby forming a seal

5.3.2. Design and Construction Considerations

A bottom seal could be created by pumping or pressure injecting grout under an existing landfill. The grout, selected for its compatibility with the wastes, would be injected through tubes placed along a predetermined grid pattern. This grid pattern would be determined by drilling exploratory boreholes to determine both vertical and horizontal limits of the sites. Grouts would be pumped to a thickness of 4 to 6 feet and would be situated about 5 feet below the site to allow for irregularities in the site bottom. Leachate collection would need to be implemented so as not to exceed capacity of the bowl (EPA, 1978).

Consideration of this method of bottom sealing should be accompanied by a great deal of caution and testing. As indicated under advantages and disadvantages of liners in Table 5-3, the method is costly and has severe technical limitations.

Another method of bottom sealing which may have limited applicability is use of a sodium bentonite slurry. American Colloid Company manufacturers a granular bentonite, which when premixed with water can be pumped into a contaminated lagoon and allowed to settle. American Colloid recommends an application rate of 2.5 - 10 lb/ft^2 of Saline Seal 100, a sodium bentonite formulation for industrial landfills or lagoons (American Colloid, 1981). This bentonite is formulated for resistance to strong concentrations of dissolved salts, acids, and alkali (Misrock, 1979). It has been used successfully to seal an oily waste treatment lagoon. This method is best suited to plugging up leaks in a lagoon bottom.

Top liners, or caps, may be used to reduce or to eliminate surface infiltration through the waste facility, and therefore to reduce the amount of leachate that is generated. Such top liners may be installed at existing sites. Various types of plastic membranes are used as top liners. A covering of soil is emplaced over the membrane to hold it in place and to protect it. Such top liners are chemically very stable and, if sufficiently protected from surface traffic, may resist breakage or puncture for years.

Clay or clayey soils may also be used to cap existing sites and to reduce infiltration. Capping soils must be carefully chosen for their low permeability characteristics and emplaced so as to provide continuous gradients away from the site. Such materials are usually rolled or otherwise compacted and may be covered by other soils to provide better trafficability in wet weather.

5.3.3 Advantages and Disadvantages of Liners as a Remedial Action for Existing Sites

Advantages and disadvantages of using liners at existing sites are summarized in Table 5.3.

5.3.4 Costs

To provide an indication of the overall costs associated with leachate collection the following example is presented, with reference to unit costs given in Table 5-4.

A disposal site has the approximate dimensions of 1,200 feet by 300 feet, and wastes have been disposed of to a depth of 12 feet below the surface.

In order to collect leachate, generated at peak rates of 50 gpm, subsurface gravity drains will be run along the perimeter of the site at a depth of approximately 18 feet. The drains will have a slight slope (~0.2%) in order that they can flow by gravity to two wetwells. Pipe placed in the drains is 8-inch perforated vitrified clay pipe, which will be imbedded in about 2 feet of gravel. In order to increase flow to the main subsurface drains, a series of 15 lateral drains (average length of 20 feet) will be

TABLE 5-3

ADVANTAGES AND DISADVANTAGES OF LINERS AT EXISTING SITES

Advantages	Disadvantages
● Can assure control of leachate migration from certain sites	● Use of prefabricated liners is technically infeasible
	● Although use of a bentonite slurry will be suitable for sealing leaks from certain surface impoundments, it is not applicable for landfills and not well suited to impoundments that are permeable along the entire site bottom
	● Construction costs for pressure injected grouts is very high
	● There may be difficulty or hazards involved in drilling through sites to inject grout
	● In using pressure-injected grout there is no available method to determine if all voids between injection points have been filled

placed at horizontal runs from the site to the main drains. The laterals will be 6-inch perforated pipe with 2 feet of gravel envelope. Four manholes have been included in the collection system to connect drains at different depths.

Leachate flowing to the wetwells will be carried by a vertical submersible pump to a leachate holding tank. Figure 5-5 depicts schematically the collection system.

TABLE 5-4

UNIT COSTS FOR A LEACHATE COLLECTION SYSTEM

Item	Unit cost	Source
Excavation; 20 ft. deep, 4 ft. wide; hydraulic backhoe	\$1.00 yd^3	1
Crushed stone; 3/4 inch Cost to buy, load, haul 2 miles, place, and spread.	\$8.30 yd^3	1
Tile Drainage Vitrified clay (Standard bell and spigot)		
4" perforated	\$2.15 LF installed	
6" perforated	\$2.63 LF installed	
8" perforated	\$4.33 LF installed	
Precast concrete manholes		
48" x 3'	\$180.59	1
48" x 4'	\$215.73	
Concrete wetwells	\$6,500.00	1
Sewer piping; Concrete; nonreinforced; extra strength		1
6" diameter	\$3.94 LF	
8" diameter	\$4.31 LF	
Bituminous fiber 4" diameter	\$2.04 LF	1
Sewer piping; PVC		2
4"	\$1.74 LF	
6"	\$2.89 LF	
8"	\$4.61 LF	
Backfilling: Spread dumped material by dozer	\$.66 yd^3	1

--continued--

TABLE 5-4 (Continued)

Item	Unit cost	Source
4" Submersible pumps installed; to 180 ft.	$1,700	1
2 HP; 840-1440GPH	$2,375	
5 HP; 1302 - 1494 GPH;		
Holding tank; Horizontal cylindrical glass fiber reinforcement phthalic resin tanks		
10,000 gal	$6,354 installed	4
20,000 gal	$14,164.50 installed	4
Portland cement grout	0.95/gallon	3
Bentonite grout	1.25/gallon	

---

[1] Godfrey, 1979
[2] McMahon and Pereira, 1980
[3] Industrial sources, 1980
[4] Richardson Engineering Services, 1980

The total volume of trenching required for the main subsurface drains is:

$$(18\ \text{ft})\ (2.67\ \text{ft})\ (3{,}000\ \text{ft}) = 144{,}200\ \text{ft}^3 \quad 5{,}334\ \text{yd}^3$$

Volume of trenching required for the laterals:

$$(18\ \text{ft})\ (2.5\ \text{ft})\ (20\ \text{ft})\ (15\ \text{drains}) = 13{,}500\ \text{ft}^3 \quad 500\ \text{yd}^3$$

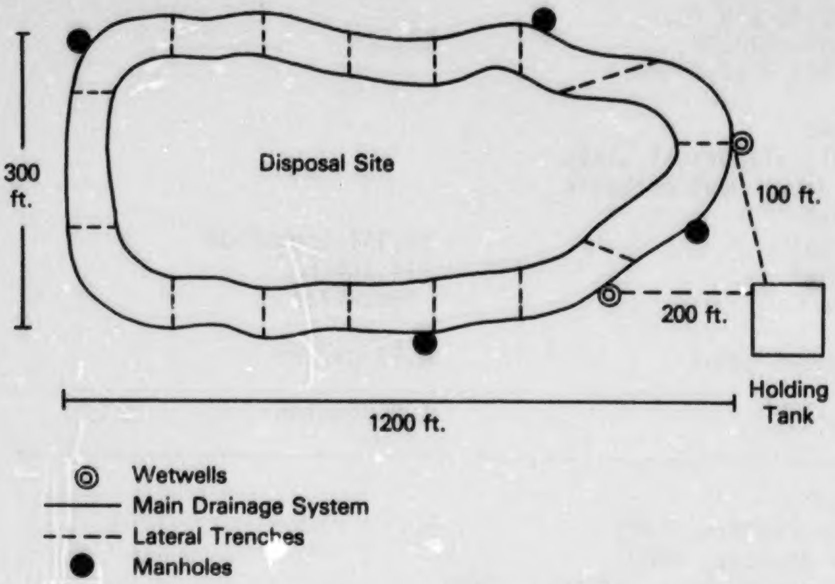
Trench excavation costs are as follows:

$$5{,}334\ \text{yd}^3 + 500\ \text{yd}^3 \times \$1/\text{yd}^3 = \$5{,}834$$

The volume of gravel required for 2 feet of envelope material is estimated as follows:

FIGURE 5-5

LEACHATE COLLECTION COSTING EXAMPLE


300 ft.
Disposal Site
100 ft.
200 ft.
1200 ft.
Holding Tank
Wetwells
Main Drainage System
Lateral Trenches
Manholes

Main subsurface drains

$[(4\ \text{ft}^2) - (.56\ \text{ft}^2)](3{,}000\ \text{ft}) = 10{,}320\ \text{ft}^3$ or $381\ \text{yd}^3$

Laterals

$[(4\ \text{ft}^2) - (.38\ \text{ft}^2)](300\ \text{ft}) = 456\ \text{ft}^3$ or $169\ \text{yd}^3$

Total cost of 3/4 inch gravel envelope is:

$[(381\ \text{yd}^3) + (169\ \text{yd}^3)](\$8.30/\text{yd}^3) = \$4{,}565$

A total length of 3,000 feet of 8-inch perforated vitrified clay pipe is required. The total is:

$(3{,}000\ \text{ft})\ (\$4.33/\text{ft}) = \$13{,}000$

A total of 300 ft of 6-inch perforated pipe is required for the laterals:

(300 ft) ($2.63/ft) = $789

The total costs for four precast concrete manholes is estimated as follows:

(4) ($215.73) = $862.93

The total cost for two concrete wetwells is:

(2) ($6,500) = $13,000

Figure 5-5 shows that 300 feet of sewer piping is required to carry the leachate to the holding tank:

(300 ft) ($3.94/ft) = $1,182.00

Two submersible pumps that can pump to 25 gpm:

(2) ($1,700) = $3,400

Total costs for materials and labor:

$5,834 + $4,565 + $13,000 + $789 + $863 + $13,000 + $1,182 + $3,400 = $42,633

5.4 LEACHATE TREATMENT

5.4.1 Leachate Treatment Modules

A leachate treatment system can be defined as a treatment module or group of modules designed to meet suitable treatment levels for stream discharge, groundwater recharge, discharge to municipal treatment systems, or recycle through a landfill.

Leachate composition and strength vary widely from landfill to landfill and within a given landfill, depending upon the nature of the waste, the age of the fill, the amount of precipitation, and the porosity, permeability and adsorption characteristics of the soil. Because of the wide variability from site to site, each treatment system must be designed on a case-by-case basis after completion of an extensive monitoring program. From a systems design standpoint, changes in leachate composition with age are problematic because a system that may be adequate now may not be suitable as the landfill ages. From an operations standpoint, fluctuations in leachate quantity and strength can cause serious problems in trying to automate chemical additions and maintain an active biomass for biological treatment. These fluctuations can be dampened to some extent by equalization, but it is evident that design and operation of an effective leachate treatment system will require a continuous monitoring program and flexibility to adjust treatment to meet changing leachate strength.

An example of the wide variability in leachate composition from site to site is presented in Table 5-5. Results of analysis from thirteen municipal sites indicate that the COD's and BOD's can vary by several orders of magnitude from one site to the next. TOC and total solids vary somewhat less, though the range of values is considerable. Extensive sampling of municipal landfills has enabled researchers to make some generalizations regarding leachate composition and age of fill; ratios of COD/TOC, BOD/COD, VS/TS, and SO_4/CL have been shown to reflect the age of the fill. Figure 5-6 shows that these ratios decrease as the landfill stabilizes, reflecting a decrease in biodegradability, an increase in washout of inorganics, and increasingly anaerobic conditions that reduce sulfates to sulfides and cause them to precipitate out as the metal sulfide (EPA, 1977a). These trends in leachate composition with age provide a useful though not infallible tool for determining changes in leachate treatment needs as a municipal landfill ages. Such trends may occur to varying extents in industrial landfills as well, depending largely on the biodegradability of the wastes. In general, industrial landfills can be expected to exhibit a lower ratio of BOD/COD where organics are part of the fill material.

Table 5-6 presents an overview of the treatment modules or unit processes which have potential for treatment of leachates of various strengths and compositions. Each unit process is summarized with regards to its applicability, major design considerations, environmental effects, technology status, and reliability. The unit processes are considered in further detail in Appendix B. Since detailed design and construction factors for these processes are well published, no attempt was made to give a comprehensive review of each process module. Rather, the objective of Appendix B is to provide sufficient information for the reader to determine feasible treatment technologies for a leachate with a particular strength and compositon. Final selection of a treatment scheme and the corrresponding design of that scheme will require thorough site investigations along with detailed engineering design and cost considerations.

Table 5-7 summarizes the relative costs and energy requirement of the various modules. In estimating costs it was assumed that low volumes of leachate would be treated at any particular site (0.1-0.3 mgd). Some of the unit processes are more cost effective for higher volumes of leachate. Costs curves are included in Appendix B and are based on 1976 dollars unless otherwise noted.

Shuckrow et al. (EPA, 1980) have also prepared a comprehensive study of unit processes suitable for hazardous waste treatment and are currently involved in studies of process train selection.

FIGURE 5-6

CHANGES IN RATIOS OF VARIOUS PARAMETERS WITH THE AGE OF THE LANDFILL

(Source: EPA, 1977)

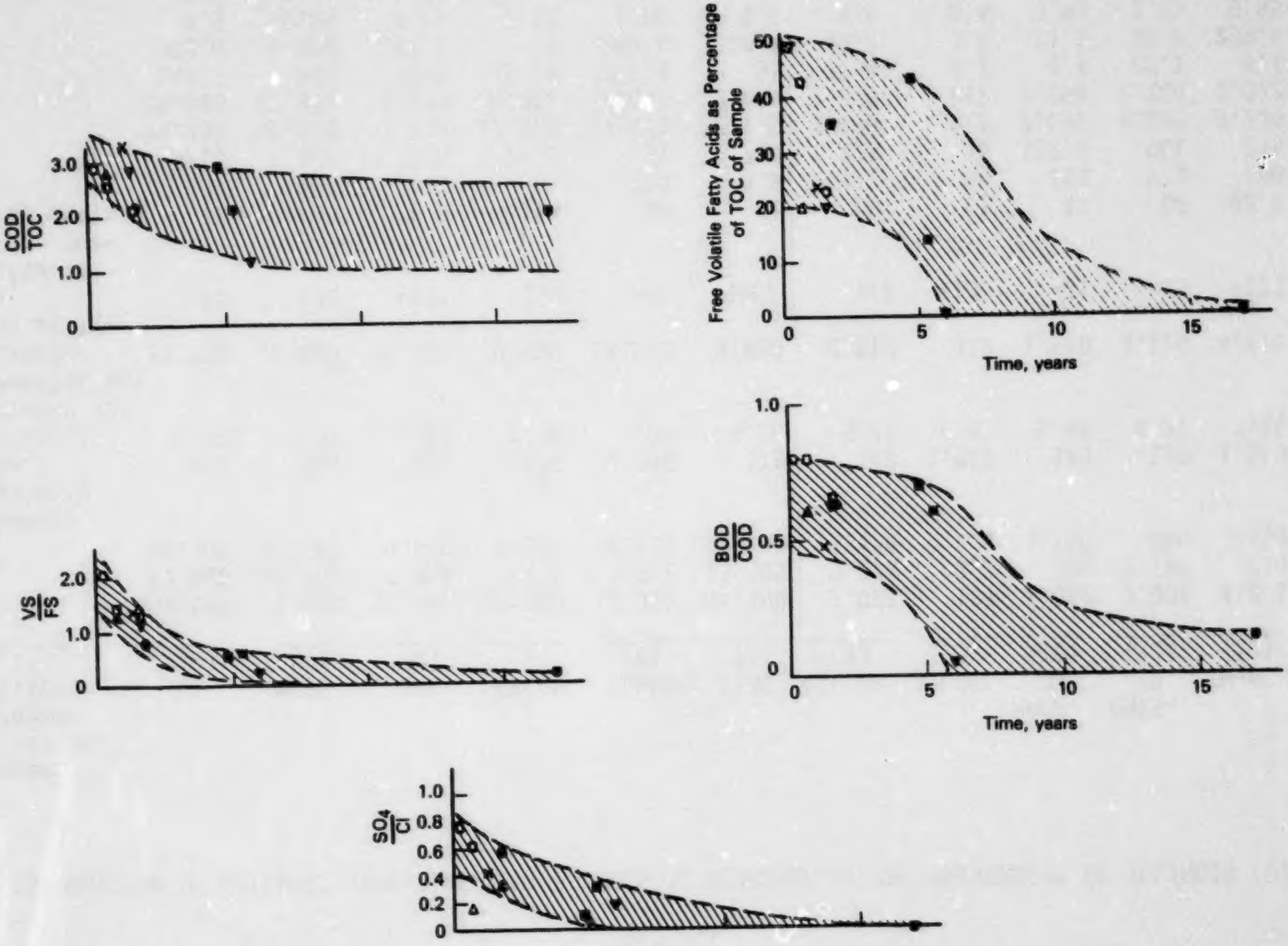


TABLE 5-5

COMPOSITION OF LEACHATE FROM DIFFERENT SOURCES AS MEASURED AT THE UNIVERSITY OF ILLINOIS (UI)

Source parameter in milligrams per liter (1)	UI (2)	MUNC (3)	UMC (4)	SONCON (5)	EDNREC (6)	CINC (7)	ATLCON (8)	ATLREC (9)	PACP-07 (10)	PHKS-12 (11)	IW-68 (12)	LW-17 (13)
COD	49.000	71.680	16.580	12.800	12.050	45.750	7.021	134	2.062	3.006	412.1	11.1
TOC	17.060	27.700	5.906	4.476	3.921	13.000	3.273	93.2	900	1.190	210	70
BOD_3	24.700	57.000	9.960	8.100	9.100	22.000	4.410	6.4	1.190	660	552	39
Bicarbonate alkalinity ($CaCo_3$)	668	569	385	2.576	1.265	175	142	1.025	149	158	1.520	1.300
pH (pH unit)	5.63	5.97	5.59	5.09	5.99	5.25	5.72	6.62	6.46	6.37	7.15	2.25
Conductance in micromilus per centimeter	13.700	16.800	5.420	10.000	12.500	9.450	2.810	978	1.540	3.115	4.515	1.400
ORP, in millivoles	-60	-132	-220	168	-95	NM[1]	+72	+130	-71	-75	+173	+97
Turbidity in Jackson turbidity units	75	80	270	110	96	71	23	61	41	58	62.5	130
SS	139	202	192	55	204	8.9	472	68	141	923	186	236
FSS	92.5	167	110	26.5	91	7.5	277	16	137.5	901	246	128
IS	33.989	55.348	7.930	12.579	14.653	32.145	4.120	911	2.091	4.225	2.704	1.45
FS	15.580	22.895	3.475	5.823	6.647	13.663	2.273	812	1.294	2.206	2.072	1.144
Org-N	544.7	945	78.5	107.4	257.4	81	27.8	4.1	4.4	20.1	8.4	3.2
NH_4-N	392.6	1.028	347.4	2	254.5	247.7	49.2	1.4	21.3	62.2	299.5	3.8
NO_3-N	0.5	10.25	4.25	7.12	1.80	9.8	6.5	0.4	0.90	1.23	0.55	0.45
NO_2-N	BDL[2]	0.04	0.04	BDL	BDL	0.19	BDL	BDL	BDL	BDL	0.05	0.06
Total-P	21.5	98	85	1.20	28.8	31.6	1.85	0.5	0.70	4.35	13.00	4.96
Ortho-P	6.5	29	85	0.80	22	211	1.7	0.25	0.55	3.4	3.08	2.70

--continued--

TABLE 5-5 (Continued)

Source parameter in milligrams per liter (1)	UI (2)	MUNC (3)	UMC (4)	SONCON (5)	EDNREC (6)	CINC (7)	ATLCON (8)	ATLREC (9)	PACP-07 (10)	PHKS-12 (11)	IW-68 (12)	LW-17 (11)
SO_4	1.110	1.558	77	181	176	909	101	30.3	19.5	29	7.4	19.5
CI	1.480	2.467	474	462	1.010	2.096	116	60.6	60.2	265	715.5	197.4
Ca	3.750	1.960	572	1.030	1.320	2.325	520	133	215	330	76	365
Mg	650	1.140	220	515	750	530	52.5	35	75	88	186	81
Me	2.200	1.046	91	301	540	735	810	0.5	560	330	4.0	1.5
Na	1.360	1.580	330	240	675	750	75	44	37.5	113	370	91
K	1.140	2.300	900	74	300	860	620	147	35	70	450	39

---

[1] Not measured

[2] Below detectable limits

(Source: EPA, 1977a)

TABLE 5-6

SUMMARY OF LEACHATE TREATMENT PROCESS UNITS

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
Activated sludge	● Can handle BODs of 10,000 ppm ● Requires low level of suspended solids - usually ≦1% ● Oil and grease should be less than 50 mg/l ● Effective for readily degradable organics or organics to which it can be acclimated (see Appendix B.3.1 and B.3.5). ● Sensitive to heavy metals (see Table B-6).	● Detention time* ● Organic load* ● Food to microorganism ratio* ● Aeration *See Table B-7	Generate excess sludge containing refractory organics that have been sorbed	Highly developed; widely used	Process reliability is very good in absence of shock loads
Ammonia stripping	● Relatively insensitive to concentration changes ● Requires pH of 10.8 to 11.5 ● Not operable at freezing temperatures	● Hydraulic load ● pH ● Temperature ● Air flow ● Tower packing and depth	May result in air pollution by NH_3 or volatile organics; not a problem if recovery is implemented	Fully demonstrated but not widely used	High if proper pH is maintained

--continued--

TABLE 5-6 (Continued)

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
Chlorination	● pH and temperature-dependent	● Temperature and pH ● Contact time ● Mixing ● Presence of interfering compounds	Can cause formation of chlorinated organics and release of chlorine gas	Fully developed and widely used	High; operation is simple
Anaerobic, Aerated Facultative Lagoons	● Requires very low suspended solids (≤ 0.1%) ● Requires low strength organic wastes (see Table B-13) ● Sensitive to heavy metals and oil and grease (see Table B-6)	● Detention time* ● Depth* ● Organic load* ● pH *See Table B-13	May create odors; may release volatile and H S and methane if anaerobic; must be lined to prevent seepage into groundwater	Well demonstrated for stabilization of organics but not widely used	High if proper pH is maintained and organic strength is low; very sensitive to shock loads since there is not sludge recycle
Biological seeding	● Sensitive to heavy metals and oil and grease ● Can degrade a wide range of organics depending	● Same as air-activated sludge or pure oxygen depending upon how the bacteria is used	Generates a sludge lower in refractory organics than conventional biological	Use of acclimated bacteria is widespread but use of commer-	Can increase reliability of conventional biologic

--continued--

TABLE 5-6 (Continued)

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
Biological seeding (continued)	upon acclimation of the culture (see Appendix B.3.5) ● Sensitive to same limitations as activated sludge with regard to metals, suspended solids, and oil and grease		treatment but sludge still contains sorbed metals	cially available cultures is not	treatment; increase stability; decrease susceptibility to shock loads
Carbon adsorption	● More amenable to treatment of certain classes of organics (see Table B-15) ● Treat organics to an upper limit of about 10,000 ppm TOC ● Oil and grease should be less than 10 ppm ● Suspended solids should be less than 50 ppm ● Where organics are being removed (see Table B-16), concentrations of less than 500 ppm are advisable	● Organic load ● Hydraulic load* ● Contact time* ● Regeneration and backwash requirements* *See Table B-18	May be emissions from thermal regeneration but these can be controlled with scrubbers or afterburners; where there is no regeneration, disposal of spent carbon can be hazardous	Highly developed and widely used	Moderate depending on design and equipment quality; insensitive to toxics but subject to clogging with high SS and oil grease; isotherms may give artificial results of process performance

--continued--

TABLE 5-6 (Continued)

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
Liquid Ion Exchange	● Volume of extractants required place a practical upper limit of about 10 g/ℓ ● Sensitive to oxidants, surfactants, and suspended solids ● Suitable for treatment of anions, cations, metal oxyanions, and weak acids	● Waste type and load ● Hydraulic load ● Mode of operation ● Regeneration requirements	Generates a highly concentrated regeneration solution; also clean stream may contain low concentrations of organic phase that should be removed		Fairly well developed but rarely used, especially for dilute waste streams
Precipitation/flocculation/Sedimentation	● No concentration limit ● Sensitive to hydraulic and waste load fluctuations ● pH-dependent	● Hydraulic load ● Concentrations of suspended solids and precipitable soluble species ● pH; alkalinity ● Settling rate ● Removal requirements for subsequent treatment processes	Generates large amounts of sludge, some of which is difficult to dewater	Highly developed; widely used	Moderate; effluent quality may vary considerably; would require frequent use of jar test to determine appropriate concentrations of flocculants and precipitants

--continued--

TABLE 5-6 (Continued)

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
Equalization	● Can accept any waste fluctuations for which it was designed	● Hydraulic flow and variations in flow ● Detention time ● Mixing ● Aeration ● Equalization demands for subsequent treatment modules	Generates sludge; potential for evaporation of volatile wastewater components	Fully demonstrated and widely used	Very high; easily designed to obtain objectives
Ion exchange	● Because of regeneration requirements, concentration should be less than 2,500 ppm (expressed as calcium carbonate) or 0.05 equiv/ℓ; may accept up to 4,000 ppm ● Suitable for treatment of all soluble metal cations or anions, anionic halides, cyanides, nitrates, and some acids ● Sensitive to suspended matter, oxidants and some organics, especially aromatics	● Waste type and load ● Hydraulic load ● Mode of operation (see Table B-21) ● Regeneration requirements (see Table B-22) ● Resin adsorption capacity	Generates a highly concentration regeneration solution that should	Fully developed but not widely used	Moderate for fixed bed; and air for continuous systems requires a high degree of monitoring and inspection

--continued--

TABLE 5-6 (Continued)

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
pH adjustment	● Amenable to wide range of conditions	● Waste type and load ● Detention time ● Mixing	May generate high concentrations of solids	Highly developed, widely used	Very high, assuming waste stream characteristics are unstable
Pure oxygen-activated sludge	● Requires suspended solid levels of about 1% or less ● Can handle higher organic loads than conventional activated sludge and is more tolerant of shock loads (see Table B-7) ● Sensitive to heavy metals and oil and grease	● Detention time* ● Organic load* ● Food to microorganism ratio* ● Oxygen requirements *See Table B-7	Generates sludge containing refractory organics and sorbed metals	Relatively new technology but demonstrated for some industrial wastewaters	Reliability is not fully established; complex operation, and requires high level of maintenance
Rotating biological disc	● Suitable for treatment of readily degradable organics; can handle higher organic loads than trickling filter but lower than activated sludge	● Detention time ● Hydraulic load ● Organic load ● Temperature	Generates sludge containing refractory organics and sorbed metals; may cause odors if	Process is relatively new, not widely but gaining in popularity	Moderate in the absence of high organic loads and temperatures >55°F

--continued--

TABLE 5-6 (Continued)

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
Rotating biological disc (continued)	● Better suited to treatment of suspended or colloidal organics rather than soluble ● As with other biological processes, sensitive to oil and grease and metals	● Number of stages and trains			
Trickling filter	● Can handle only very low organic loads as compared to activated sludge ● Better suited to treating suspended and colloidal organics rather than soluble ones ● As with other biological processes, sensitive to metals and oil and grease	● Media type* ● Hydraulic load* ● Organic load* ● Bed depth* ● Temperature ● Recirculation *See Table B-9	Generates sludge that contains refractory organics and sorbed metals; causes odors	Widely used as a roughing filter for industrial wastes	Fair for secondary treatment; moderate as a roughing filter
Wet air oxidation	● Solution of any organics can be oxidized if temperature and pressure are high enough	● Temperature ● Residence time ● Waste COD	Process is very clean and generates no air pollution	Relatively new process and not fully demonstrated	Process reliability has not been adequately

--continued--

TABLE 5-6 (Continued)

Treatment method	Feed stream requirements and limitations	Major design and performance criteria	Environmental impact	Technology status	Reliability
	● Range of organics treated is about 5-150 g/ℓ				tested, process is complex and requires use of controls

Sources: Liptak, 1974; EPA, 1979; DeRenzo, 1978; Hammer, 1975

TABLE 5-7

COST COMPARISON FOR TREATMENT MODULES (0.1 AND 0.3 mgd)

Process	Capital Cost[1]	O&M[1]	Relative Energy Use[1]
Activated sludge (with clarifier)	High	Low	Moderate
Ammonia stripping (no recovery)	Low/ moderate	High	High
Anaerobic/facultative lagoons	High	Low	Low
Biological seeding (used with conventional or pure oxygen-activated sludge)	High	Low to moderate	Moderate to high
Carbon adsorption (no regeneration)	Moderate	Moderate to high	Low
Chlorination	Low	Low	High
Equalization	Low	Low	High
Ion exchange	Generally high but very variable	Moderate to high, variable	Low
Liquid ion exchange	Generally high but very variable	Moderate to high, variable	Low
Precipitation/floc/sedimentation	Low	Moderate	Low
Pure oxygen-activated sludge (including clarifier)	Moderate/ high	Moderate	Moderate
Rotating biological disc (including clarifier)	Moderate/ high	Moderate	Moderate
Trickling filter	Moderate	Low	Low
Wet air oxidation	High	High	High

[1] See legend on next page.

Source: EPA, 1978; DeRenzo, 1978; EPA, 1979

TABLE 5-7 (Continued)

Legend Explaining Relative Cost Comparisons

(Based on 1978-79 costs)

Capital Costs

Low: <$75,000 for 0.1 mgd;
<$100,000 for 0.3 mgd

Moderate: $75,000 - $110,000 for 0.1 mgd;
$100,000 - $200,000 for 0.3 mgd

High: <$110,000 for 0.1 mgd;
<$200,000 for 0.3 mgd

O&M Costs

Low: <$7,500 for 0.1 mgd;
<$1,300 for 0.3 mgd

Moderate: $7,500 to $20,000 for 0.1 mgd;
$1,300 to $25,000 for 0.3 mgd

High: <$20,000 for 0.1 mgd;
<$25,000 for 0.3 mgd

Energy

Low: <7% of O&M

Moderate: 7-15% of O&M

High: <15% of O&M

5.4.2 Leachate Treatment System Cost Example

Although the costs associated with individual treatment modules have been provided in Appendix B, it is evident that leachate treatment will require a series of several modules in order to meet treatment objectives in most cases.

This section provides treatment costs for a leachate treatment system designed for a municipal landfill. Although the treatment system is typical of what one might expect for the wastes specified, the wastes (and treatment components) are not representative of all landfills. Each site will be unique in terms of its treatment needs.

The cost example for the municipal leachate treatment system is based on the leachate characteristics of the GROWS Landfill in Bucks County, Pennsylvania, and on the basic design for that treatment system (EPA, 1977b). Cost curves presented throughout Appendix B are used as the basis for cost estimates, rather than actual costs incurred by construction of the GROWS treatment system.

The GROWS Landfill is an active municipal landfill that generated a maximum leachate flow of about 0.31 mgd during an operating period when influent and effluent had the characteristics shown in Table 5-8.

The treatment system is illustrated in Figure 5-7 and consists of lime addition, sedimentation, air stripping, neutralization, and activated sludge treatment.

FIGURE 5-7

SCHEMATIC OF LEACHATE TREATMENT SYSTEM AT GROWS LANDFILL

(Source: EPA, 1977)

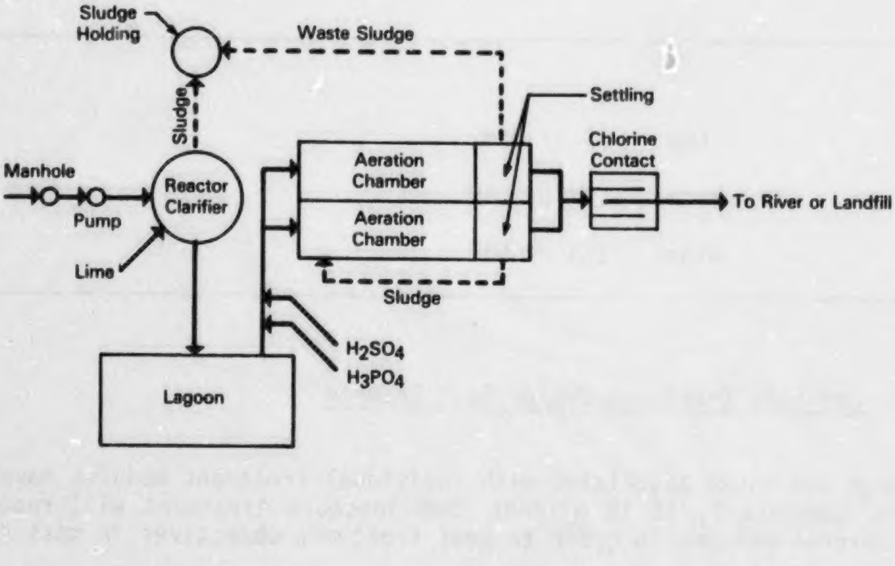

Sludge Holding
Waste Sludge
Settling
Sludge
Chlorine Contact
Manhole
Pump
Reactor Clarifier
Aeration Chamber
Aeration Chamber
To River or Landfill
Lime
Sludge
H_2SO_4
H_3PO_4
Lagoon

TABLE 5-8

TREATMENT PERFORMANCE AFTER ACCLIMATION OF ACTIVATED SLUDGE

	Concentration		
Item	Influent	Effluent	Percentage removal
Suspended solids	445	126	71.7
Dissolved solids	10849	5369	50.5
COD	9689	576	94.1
BOD	4993	60.5	98.8
Alkalinity	3718	388	89.6
Hardness	4647	1629	64.9
Magnesium	495	109	78.0
Calcium	819	472	42.4
Chloride	3172	2925	7.8
Sulfate	197	1333	--
Phosphate	1.62	17.8	--
Ammonia-N	510	46.5	90.9
Kjeldahl-N	539	141	73.8
Sodium	992	724	49.1
Potassium	823	505	38.6
Cadmium	0.049	0.014	71.4
Chromium	0.105	.075	28.6
Copper	.313	.078	75.1
Iron	205	.96	99.5
Nickel	.52	.27	48.1
Lead	.545	.12	78.0
Zinc	3.64	.44	87.9
Mercury	.015	.004	73.3

Source: EPA, 1977

Treatment design criteria are as follows:

Lime Clarification

The primary circular clarifier is designed for an overflow rate of 800 gal/d/ft^2. Costs include sludge pumps. Lime storage and feed equipment are based on the use of hydrated lime at dosages of 440 - 6000 mg/ℓ which are required to raise the pH above 10 for ammonia stripping.

Ammonia Stripping

Although the GROWS landfill uses an ammonia stripping lagoon, cost estimates have been made for a packed tower. Ammonia recovery was not included; although the ammonia concentrations are high, flow is relatively low. Construction costs include a 20-foot-high packed tower, packed with 1/2-inch diameter PVC pipe at 3 inch centers. Hydraulic loading to the tower is 1.0 gal/min/ft^2 and the air/water ratio is 400 ft^3/gal.

Neutralization

Prior to discharging to activated sludge treatment, the effluent from ammonia stripping is adjusted to neutral pH with H_2SO_4; pH adjustment requires 0.69 gal H_2SO_4/1000 gal.

Activated Sludge

Biological treatment consists of two aeration units and two secondary clarifiers in sequence. Experience at the GROWS landfill indicated that it was necessary to maintain a MLVSS concentration of 3000-8000 mg/ℓ, in order to obtain 90 percent BOD reduction for the high strength leachate. This high MLVSS concentration was maintained by return sludge pumps capable of delivering a return sludge flow equal to 200 percent of the influent flow. Units could operate as conventional sludge or extended aeration units. Extended aeration was chosen for costing purposes and included a chlorine contact chamber. It was also their experience at GROWS that the sludge required additional phosphorous as a nutrient supplement. Phosphoric acid was added at a rate of 0.085 gal/1000 gallons. Separation of treated wastewater from the MLVSS was achieved by gravity sedimentation in secondary clarifiers.

Costs for this system are summarized in Table 5-9.

5.4.3 Leachate Recycle

Leachate recycle involves the recirculation of leachate back through the refuse pile for purposes of accelerating the rate of biological stabilization and in order to remove readily degradable pollutants from the leachate. Leachate concentrations are reduced by anaerobic degradation.

Leachate recycling is applicable to leachate with a high readily degradable organic content. The ratio of BOD to COD should be high for successful recycling. The process is not well suited to most industrial waste fills.

TABLE 5-9

COSTS FOR MUNICIPAL LEACHATE TREATMENT SYSTEM

Item	Expenditure	Cost
Primary clarifier	Construction	$ 69,300
	Annual O&M	4,000
Lime storage and feed	Construction	14,000
Lime: @ $1.20/1000 gal*	Materials	135,000
Ammonia stripping tower	Construction	140,000
	Annual O&M	46,000
Neutralization with H_2SO ; @ 64¢/1000 gal	Annual materials	7,000
Nutrient addition; phosphoric acid @ 26¢/1000 gal	Annual materials	1,300
Package plant; activated sludge; extended aeration; 2 stages; includes chlorination and secondary clarification	Construction	460,000
	Annual O&M	62,000
TOTAL	Construction	$683,000
	O&M	$255,300

---

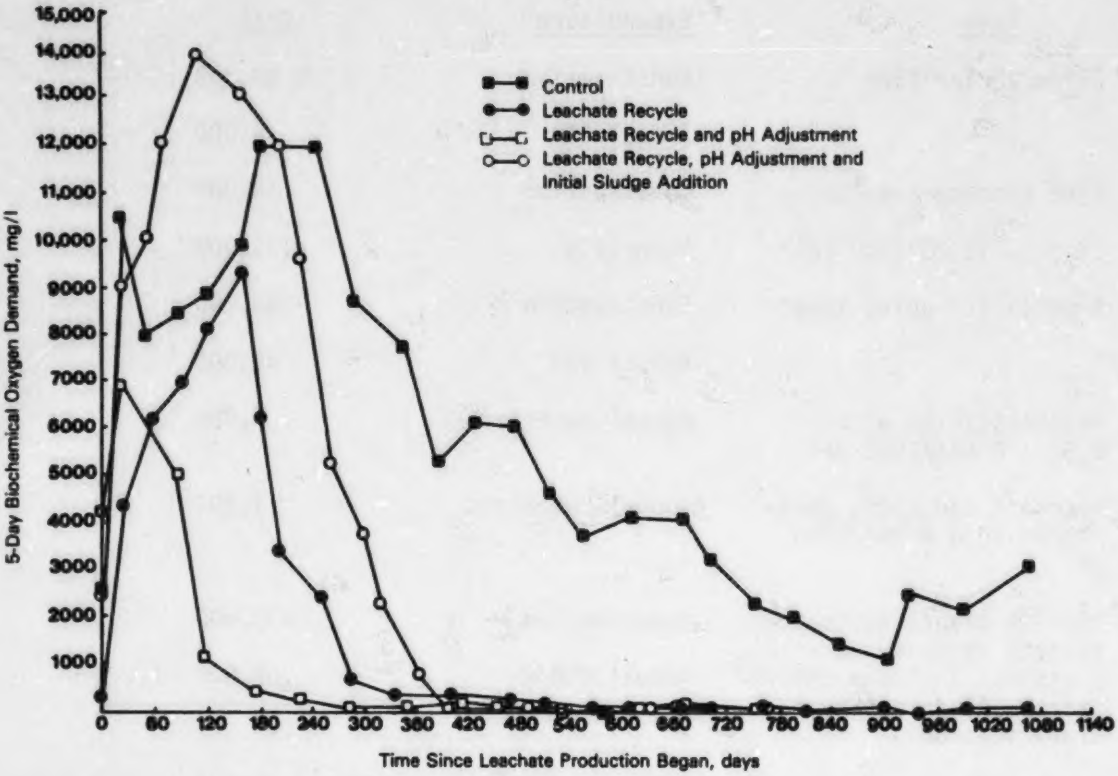
* Even higher lime dosages were required during some periods of the demonstration.

Leachate recycling is a relatively recent development and is currently not widely practiced. A recently completed leachate recycling demonstration project can provide some indication of the process efficiency and requirements (Pohland, 1975). As shown in Figures 5-8 through 5-10, a relatively high strength municipal leachate was generated from the fill used for this demonstration. Leachate recycle with and without pH adjustment and pH adjustment plus sludge addition were compared to a control. As shown in Figures 5-8

FIGURE 5-8

BIOCHEMICAL OXYGEN DEMAND OF LEACHATE

(Source: EPA, 1975)



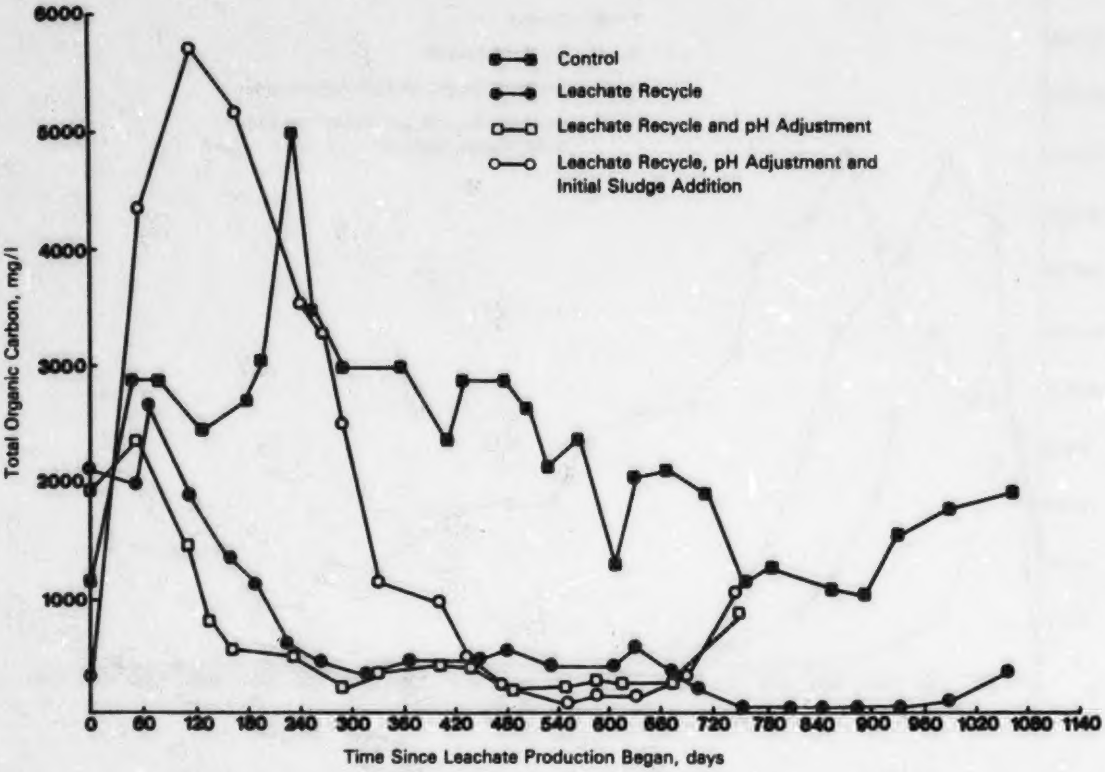
through 5-10, BOD, COD, and TOC all exhibit the same pattern: low residuals were obtained in a short time relative to the system without leachate recycle. Recycling promotes a more rapid development of anaerobic activity and methane fermentation, and it increases the rate and predictability of biological stabilization of readily available organics. Control of pH and initial sludge seeding can further enhance these effects. Optimum pH for stabilization of an anaerobic culture is 6.8 to 7.2, since it is in this pH range that methane-forming bacteria are most active.

The concentrations of most heavy metals in the leachate were low at the start of the project. However, the results generally indicated that there may be an initial increase in the levels of metals. Initially, the fill is reduced and metals are precipitated as the sulfide. With an increase in volatile acids, the environment becomes less reducing, the sulfides are oxidized, and the metals are released.

FIGURE 5-9

TOTAL ORGANIC CARBON CONCENTRATION OF LEACHATE

(Source: EPA, 1975)

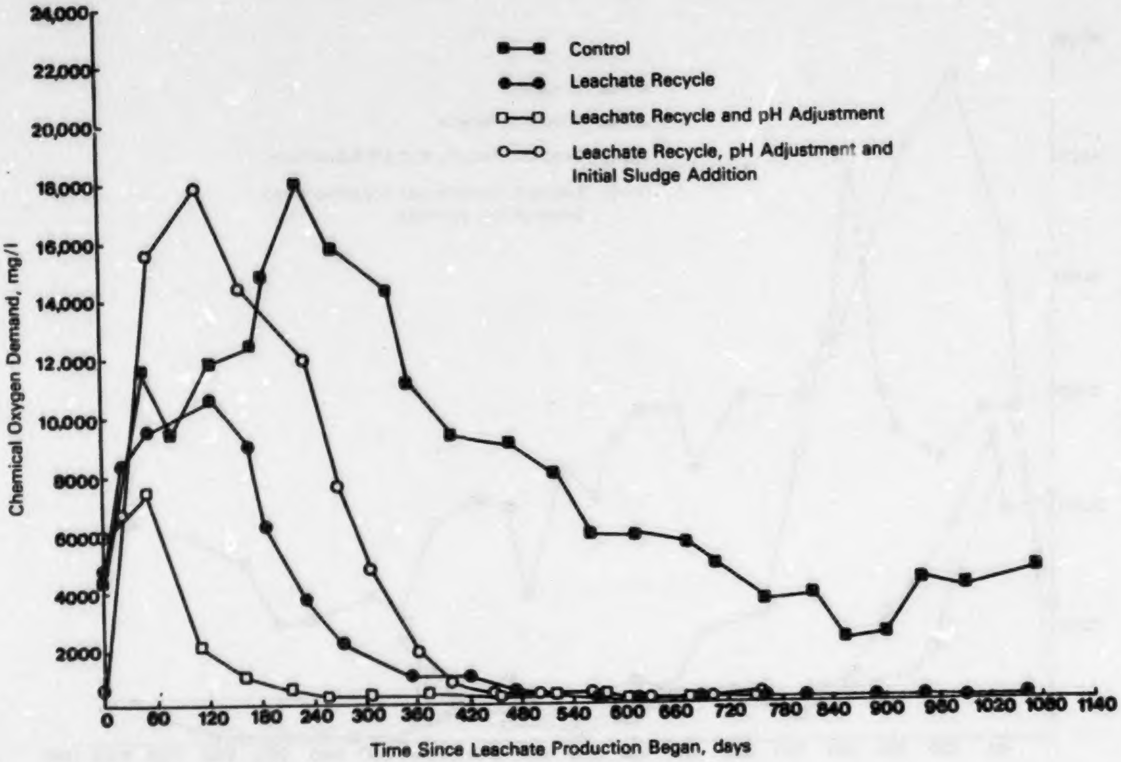


Recirculation rates and nutrient requirements will need to be determined on a case-by-case basis. The demonstration project discussed above used recirculation rates on the order of 0.5-2.0 gal/ft^2/day for a municipal refuse that was manually compacted to a density of 537 lb/yd^3.

FIGURE 5-10

CHEMICAL OXYGEN DEMAND OF LEACHATE

(Source: EPA, 1975)



REFERENCES

American Colloid Co. 1981. Volclay product information. Skokie, IL.

Baver, L. D. et al. 1972. Soil physics. New York: John Wiley and Sons, Inc.

Bern, J. 1976. Living with Leachate. In: Proceedings of the fourth national congress. Waste management technology and resources and energy recovery. Atlanta. November 12-14, 1975.

De Renzo, D.J. 1978. Unit operations for treatment of hazardous industrial wastes. Noyes Data Corp., Park Ridge, N.J.

Freeze, R.A., and J.A. Cherry. 1979. Groundwater. Englewood Cliffs, NJ: Prentice-Hall.

Godfrey, R. (ed.) 1979 Building construction cost data, 1980. Kingston, MA: R. S. Means Company, Inc.

Glaubinger, R. S. et al. 1979. Love canal aftermath. Chemical Engineering. Volume 86. No. 23, 86-92.

Hammer, M. J. 1975. Water and wastewater technology. New York: John Wiley & Sons, Inc.

Linsley, R., and J. Franzini. 1979. Water resources engineering, 3d ed. New York: McGraw-Hill Book Company.

Liptak, B. G., ed. 1974. Environmental engineers' handbook, vol. 1: Water pollution. Radnor, PA: Chilton Book Company.

McMahon, L., and P. Pereira. 1979. 1980 Dodge guide to public works and heavy construction costs. New York: McGraw-Hill Information Systems Co.

Misrock, A. Atlas-Misrock Co., Inc. May 1980. Personal communication with K. Wagner.

Pohland, T.G. 1979. Pilot scale investigation of accelerated landfill stabilization with leachate recycle. Proceedings of the Fifth Annual Research Symposium - Municipal Solid Waste Land Disposal. EPA-600/9-79-023a.

Richardson Engineering Services. 1980. Richardson rapid construction cost estimating system, vol. 4: Process plant construction estimating standards.

Slover, E. 1976. A case history: Implementing a chemical waste landfill. IN: Proceedings of the fourth national congress, waste management technology and resource and energy recovery, Atlanta, 12-14 November 1975.

Solid Waste Management. 1979. Oregon landfill seeks to control leachate under difficult conditions, Solid Waste Management 9:21.

Thornthwaite, C.W., and J.R. Mather. 1957. Instructions and tables for computing potential evapotransperation and the water balance. Drexel Institute Publications in Climatology 10(3):185-311.

U.S. Environmental Protection Agency. 1977. Demonstration of a leachate treatment plant. Office of Solid Waste. Washington, D.C. PB-269-502.

U.S. Environmental Protection Agency. 1977a. Evaluation of leachate treatment. E.S. Chian and F.B. Dewalle for Municipal Environmental Research Laboratory. Cincinnati, OH. EPA-600/2-77-186a+b.

U.S. Environmental Protection Agency. 1978. Innovative and alternative technology assessment manual. Office of Water Program Activities. Washington, D.C. EPA -430/9-78-009.

U.S. Environmental Protection Agency. 1978. State-of-art study of land impoundment techniques. W.S. Stewart, Exon Research and Engineering Co. for Municipal Environmental Research Laboratory. Cincinnati, OH. EPA-600/2-78-196.

U.S. Environmental Protection Agency. 1979. Design and construction of covers for solid waste landfills. R.J. Lutton et al. for U.S. Army Engineering Experimental Station and Municipal Environmental Research Laboratory. Cincinnati, OH. EPA-600/2-279-165.

U.S. Environmental Protection Agency. 1979. Estimated waste treatment costs, vol. 3: Cost curves applicable to 2500 gpd. to 1.0 mgd. treatment plants. Municipal Environmental Research Laboratory. Cincinnati, OH. EPA-600/2-79-162c.

U.S. Environmental Protection Agency. 1980. Management of hazardous waste leachate. R.J. Shuckrow et al., for Municipal Environmental Research Laboratory. Cincinnati, OH. SW-871.

Van Schlifgaarde, J. 1974. Drainage for agriculture. American Society of Agronomy. Agronomy 17. Madison, WI.

6.0 GAS MIGRATION CONTROL

Approaches to control of gas from landfilled materials can be grouped into two categories: control of methane, and control of volatile toxics.

Control of methane gas is important at sites where biodegradable organics are present. Anaerobic decomposition of organics produces methane gas, which forms an ignitable mixture with air at concentrations of from 5 to 15 percent (Moore and Rai, 1977). Methane diffuses readily along paths of least resistance and may travel laterally and collect in underground structures, thus presenting an imminent hazard. Methane is not usually an explosion hazard in the soil, since its concentration is usually much greater than the upper explosive limit. Approaches to the control of methane migration are aimed principally at stopping lateral subsurface migration rather than controlling emissions to the atmosphere.

Control of volatile toxic compounds, on the other hand, is concurrently aimed at limiting both the lateral movement and atmospheric emissions of toxic vapors. Gas collection and emission control treatment are mandatory for volatile toxics, while their use in sites generating methane is primarily for fuel recovery.

Before gas migration controls can be properly installed at a hazardous waste site, it is important to determine the type of wastes present, the depth of fill, and the subsurface geology of the site and adjacent areas. Also, field measurements to determine gas concentrations, positive or negative pressures, and soil permeabilities should be used to establish optimum design of vent systems.

6.1 PIPE VENTS

6.1.1 General Description

Pipe vents consist of vertical or lateral perforated pipe installed in the landfill for collecting gases or vapors. They may be installed in and around the landfill alone, or in combination with trench vents (see Section 6.3) for the control of lateral gas migration. Pipe vents are usually surrounded by a layer of coarse gravel to prevent clogging by solids or water. They may discharge directly to the atmosphere or be connected to a negative pressure collection system (see Section 6.4). Various configurations of pipe vents are shown in Figure 6-1.

6.1.2 Application

Pipe vents can be employed to control lateral and vertical migration for both methane and volatile toxics. The basic configurations in Figure 6-1 cover, or can be modified to cover, most of these applications. Atmospheric vents, both mushroom and "U" type, are used for venting methane at points where gas is collecting and building up pressure. Atmospheric vents can be placed at strategic locations where sampling (with gas probe) has detected an area of gas collection. Methane will be vented to the atmosphere when the absolute pressure adjacent to the gas vent is higher than the barometric pressure. The maximum pressure differential is expected to be only a fraction of an inch of water. The use of atmospheric vertical pipe vents, without other measures to control methane gas, is limited to sites where lateral migration is restricted by impermeable strata, and where gases are collecting in a centralized area such as the crown of a landfill. Control of lateral migration of methane by an array of atmospheric pipe vents surrounding a landfill is believed to have little success unless vents are located very close together (Moore, 1976). Such a situation approaches a trench vent, to be discussed later in Section 6.2.

Methane migration control can be more effectively accomplished by installing forced-ventilation systems in which a vacuum pump or blower is connected to the discharge end of the vent pipe. A drawdown with a radius of influence of 150 feet is accomplished with a pumping rate of 50 ft^3/min (Carlson, 1977). Such a system is applicable for controlling both vertical and lateral movement of methane in the landfill, by installing vents inside and along the perimeter of the site. The collected methane can be vented to the atmosphere, flared, or recovered as a low-grade fuel gas.

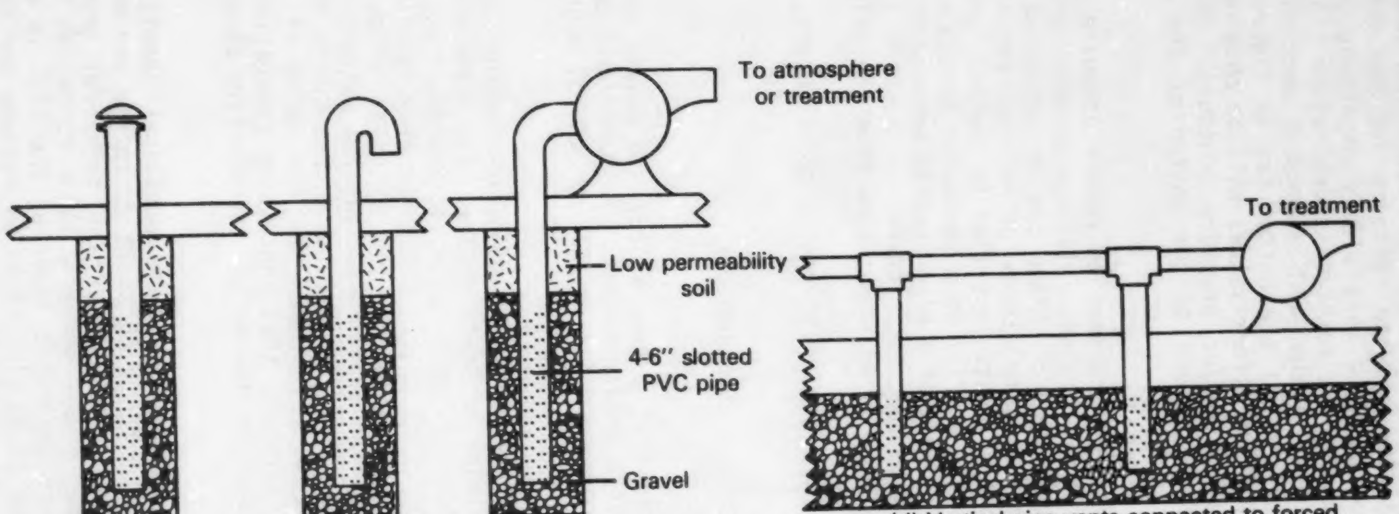
In landfills containing volatile toxics, a closed forced-ventilation system is required to prevent any toxic vapors from migrating laterally or vertically through the cover material to the atmosphere. Drawing (d) in Figure 6-1 depicts a series of pipe vents connected to a manifold that leads to a blower and finally to gas treatment. Such a configuration can be used to prevent emission of toxics to the atmosphere across the entire area of the site.

6.1.3 Design and Construction Considerations

When designing installations of atmospheric pipe vents for methane control, proper placement is the chief consideration. Preliminary sampling

FIGURE 6-1

DESIGN CONFIGURATION OF PIPE VENTS


To atmosphere
or treatment
To treatment
Low permeability
soil
4-6" slotted
PVC pipe
Gravel

(a) Atmospheric vent Mushroom Top

(b) Atmospheric vent "U" Top

(c) Forced Ventilation

(d) Vertical pipe vents connected to forced ventilation manifold system should be conducted to determine gas collection points for proper vent placement. Figures 6-2 and 6-3 show methane contours for two landfills. It is apparent that methane concentrations vary widely depending on the specific landfill configuration. The highest methane concentration (70 percent is the theoretical limit) is expected in the most anaerobic section of the filled material. In many cases, this is at the bottom (as in Figure 6-2), but not necessarily (Figure 6-3). Optimum effectiveness will be obtained if vents are placed at maximum concentration and/or pressure contours. To ensure proper ventilation, vent depth should extend to the bottom of the fill material.

Proper spacing of vents is important to ensure adequate ventilation of large areas where methane is concentrated. Distance between vents will depend on soil permeability; however, this distance can be estimated for a typical soil. Moore (1976) has derived predictions of methane reduction resulting from a series of atmospheric pipe vents installed to control lateral migration around a hypothetical municipal landfill in a permeable strata with a porosity of 0.4. If the radius of influence of a pipe vent is assumed to extend to the elbow of the concentration reduction ratio curve, as shown in Figure 6-4, the radius of influence of the vent can be calculated from the value r/r_f where:

r_f = the radius of landfill is 160 m. The radius of influence, r, then would be:

$$r = 0.125 \times 160 \text{ m} = 20 \text{ m (66 ft)}.$$

This calculation suggests that a general rule to ensure adequate ventilation would be to locate wells 50 feet apart. As mentioned earlier, atmospheric vent wells are not recommended for control of lateral migration of gas.

Forced ventilation is a more effective means of controlling the lateral and vertical migration of methane or toxic vapors. The flow rate for venting should be high enough to collect all gases being generated, i.e., it should be at least equal to the gas generation rate. Also, the flow rate should be high enough to ensure a fairly large radius of influence, so as to minimize the number of wells needed to vent the area. When venting methane for recovery, the flow rate must also be low enough so that no excess air is drawn into the system. Excessive in-flow could (1) reduce the BTU value of the gas; (2) cause the landfill to become aerobic, thus inhibiting generation of methane; and (3) cause spontaneous combustion due to the introduction of oxygen.

Studies of gas production rates at three municipal landfills in California indicated a range of from 22 to 45 ml/kg of refuse per day (Constable et al., 1979). Assuming a bulk density of 250 kg/m^3 for ground domestic garbage (Liptak, 1974), these values convert to a range of 5.5 to 11.25 liters/ m^3/day. If the average anaerobic layer of the fill is assumed to be 10 meters, then one can expect 55 to 113 liters of methane per day per square meter of fill area. This translates to a ventilation requirement of at least 6 to 11 cubic feet per minute per acre. In an actual demonstration for recovering methane from a municipal landfill, a steady state flow was obtained at 50 ft^3/min with the radius of influence at about 130 feet. This translates to a ventilation rate of 107 ft^3/ min/acre, which means a substantial portion of excess air was introduced into the system (Carlson, 1977). However, it was determined that methane production was not inhibited by this amount of air, and maximum oxygen levels in the gas were only 4 percent.

Diffusion rates of volatile toxics should be calculated to determine requirements for ventilation of hazardous waste landfills. Some research has been done on predicting vapor flux through soils. Farmer et al., (1978) developed an equation for predicting vapor flux through soil based on diffusivity, as shown below.

$$J = D_o(P_a^{10/3}/P_t^2)(C_2 \quad C_s)/L$$

where J = vapor flux from soil surface ($ng/cm^2/day$)

D_o = vapor diffusion coefficient in air ($cm^{2/day}$)

P_a = soil air-filled porosity (cm^3/cm^3)

P_t = total soil porosity (cm^3/cm^3)

C_2 = concentration of the volatilizing material at the surface of the soil
($\mu g/1$)

C_s = concentration of the volatilizing material at the bottom of the soil
($\mu g/1$)

L = soil depth (cm)

The concentration of vapor at the bottom of the soil layer can be determined from the vapor pressure of the volatile substance, as follows (Farmer et al., 1978):

$$C_s = pM/RT$$

where p = vapor pressure (mmHg)

M = molecular weight of compound (g/mole)

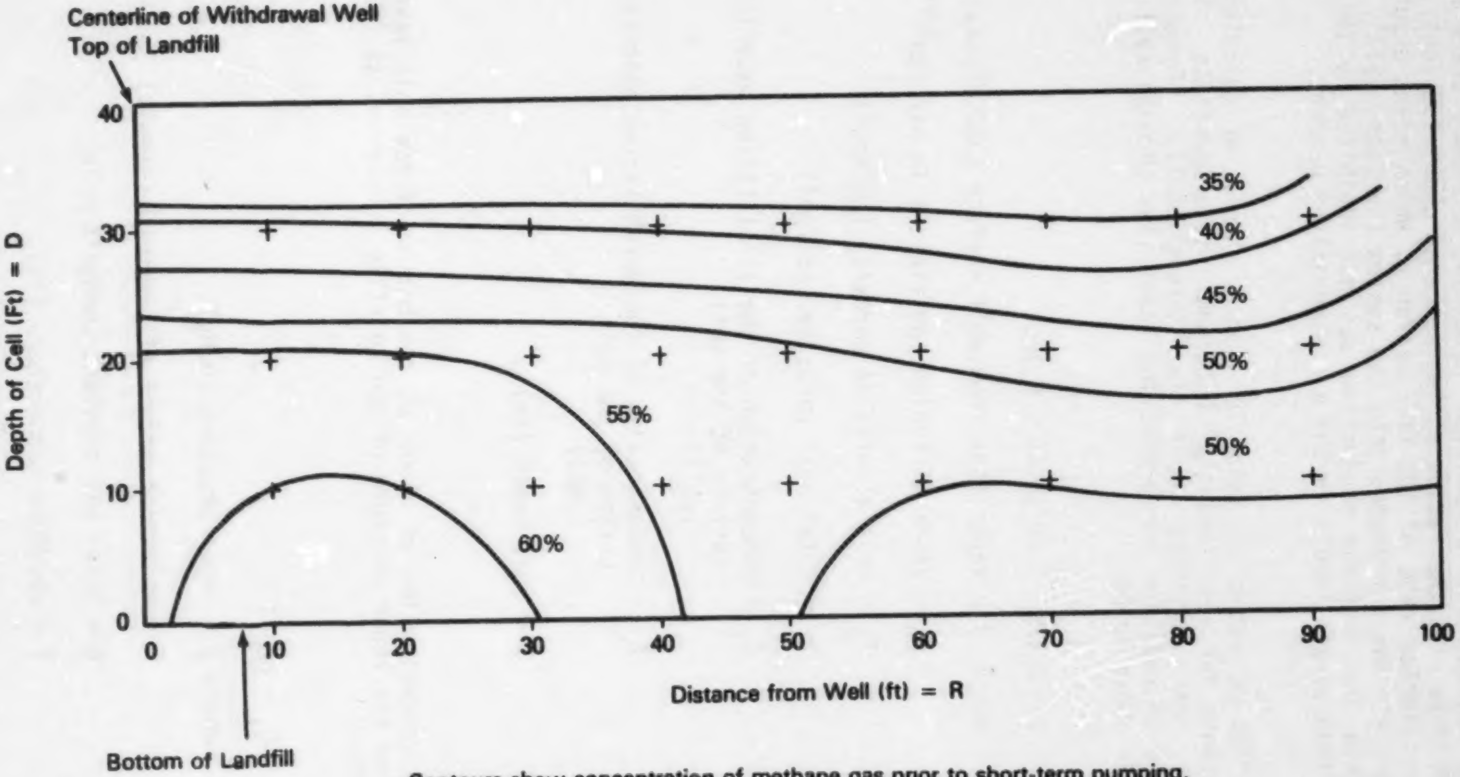
R = molar gas constant (mmHg/°K mole)

T = absolute temperature (°K)

FIGURE 6-2

UNIFORM DISTRIBUTION OF METHANE GAS IN LANDFILL

(Carlson, 1977)



Contours show concentration of methane gas prior to short-term pumping.

FIGURE 6-3

NON-UNIFORM DISTRIBUTION OF METHANE GAS IN LANDFILL

(Source: Constable et al., 1979)

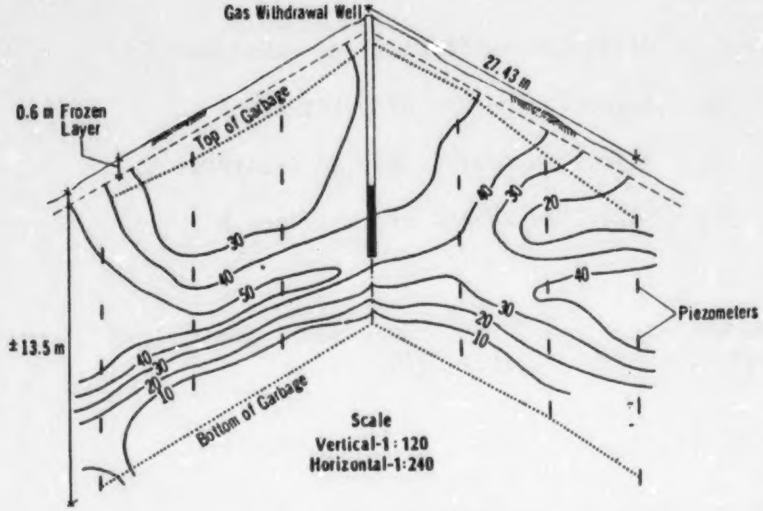
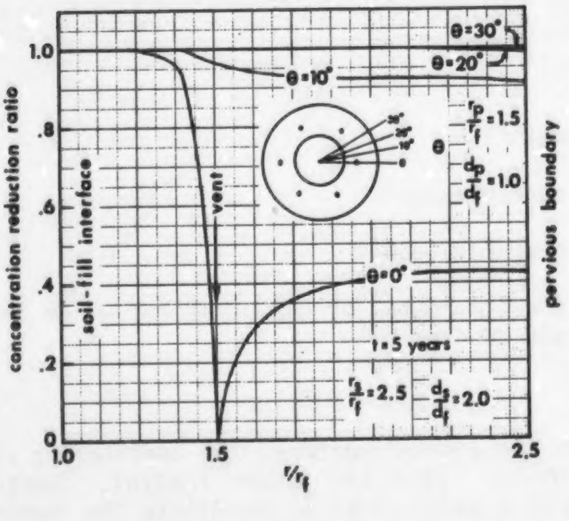

Gas Withdrawal Well
27.43 m
0.6 m Frozen Layer
Top of Garbage
30
40
50
40
30
20
10
20
30
40
30
20
10
Piezometers
± 13.5 m
Bottom of Garbage
Scale
Vertical-1:120
Horizontal-1:240

FIGURE 6-4

VARIATION OF EFFECTIVENESS OF VENTS

(Source: Moore, 1976)



Vapor pressures are available for most organic substances. Diffusion coefficients are available for a very limited number of substances, and may be estimated from a known vapor diffusion coefficient from another substance by the following formula (Farmer et al., 1978).

$$D_A = D_B(M_B/M_A)^{\frac{1}{2}}$$

where D_A = diffusion coefficient of substance A

M_A = molecular weight of substance A

D_B = diffusion coefficient of substance B

M_B = molecular weight of substance B

The diffusion coefficient increases with temperature according to the following operation (Farmer et al., 1978).

$$D_2 = D_1\ (T_2/T_1)^{\frac{1}{2}}$$

where T = absolute temperature (°K)

Mass flux rates through soil have been determined for a few organic substances, such as chloroform and hexachlorobenzene (Shen and Tofflemire, 1980). A rough approximation of flux rates for other volatiles under similar soil conditions can be calculated as follows (Shen and Tofflemire, 1980).

$$E = \frac{p \times (M)^{\frac{1}{2}} \times E}{P_i \times (M_i)^{\frac{1}{2}}}$$

where E = mass flux

p = vapor pressure

M = molecular weight

i = refers to data for chemicals for which flux rate is available

As mentioned with the case of methane, the ventilation rate should exceed the mass flux rate of the vapor for proper control. Ventilation rates for venting toxics should not be so high as to dilute the vapor stream unnecesarily, thereby raising pumping and gas treatment costs.

Some information has been developed with respect to withdrawal rates from vents and head loss (Carlson, 1977). Figure 6-5 shows head versus withdrawal rates for a gas vent withdrawing at the bottom of a landfill of compacted refuse. Figure 6-6 shows head loss as a function of distance from wells. Carlson defines the radius of influence as the distance where the head is -0.1

FIGURE 6-5

DISCHARGE RATING CURVE

(Source: Carlson, 1977)

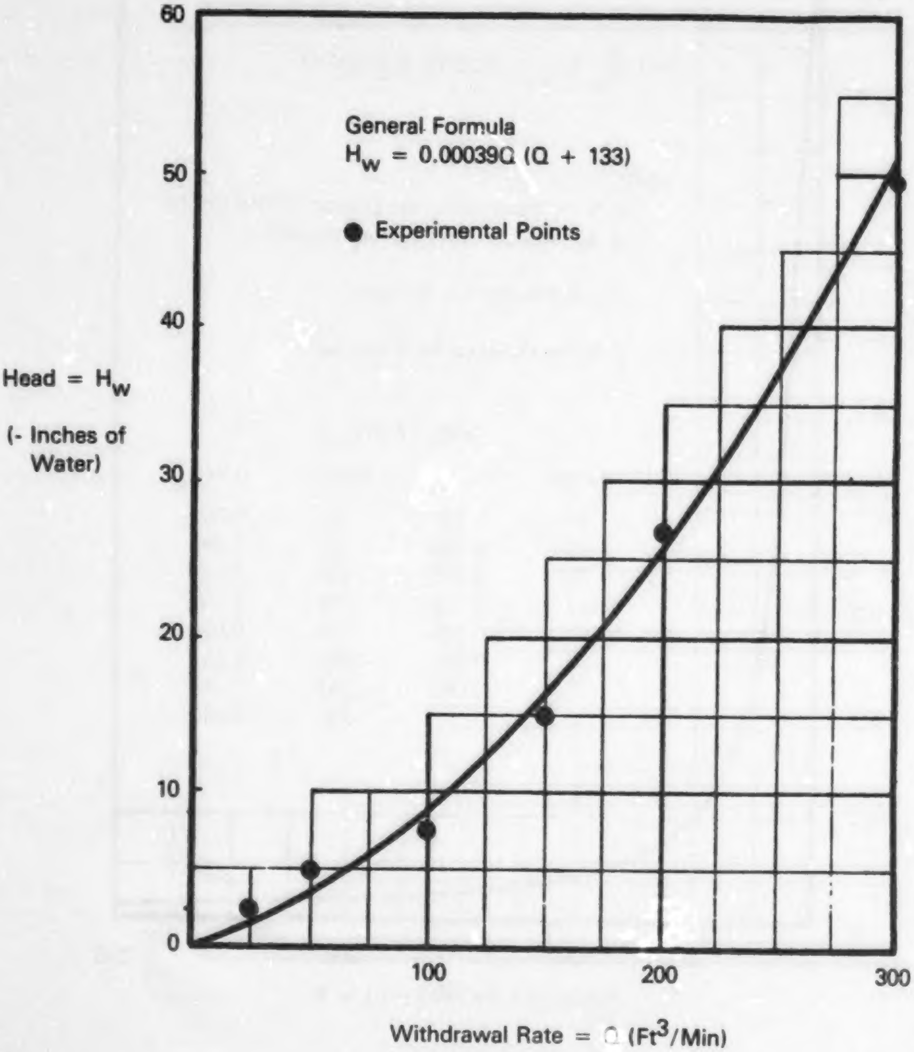
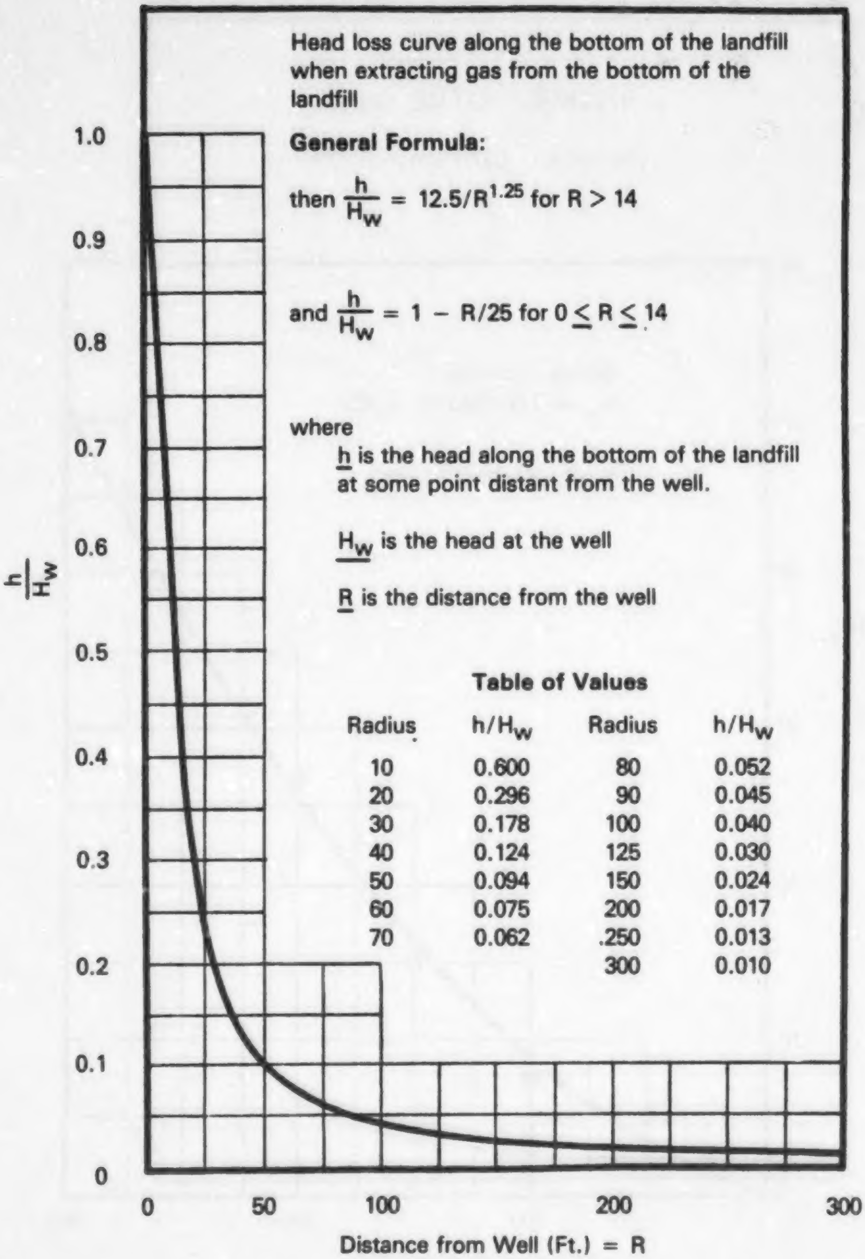


FIGURE 6-6

HEAD LOSS CURVE

(Source: Carlson, 1977)



Head loss curve along the bottom of the landfill when extracting gas from the bottom of the landfill

General Formula:

then $\frac{h}{H_w} = 12.5/R^{1.25}$ for $R > 14$

and $\frac{h}{H_w} = 1 - R/25$ for $0 \leq R \leq 14$

where

$\underline{h}$ is the head along the bottom of the landfill at some point distant from the well.

$\underline{H_w}$ is the head at the well

$\underline{R}$ is the distance from the well

Table of Values

Radius	h/H_w	Radius	h/H_w
10	0.600	80	0.052
20	0.296	90	0.045
30	0.178	100	0.040
40	0.124	125	0.030
50	0.094	150	0.024
60	0.075	200	0.017
70	0.062	.250	0.013
		300	0.010

inches of water, which is the point at which changes in atmospheric pressure would predominate. These figures are related to a specific landfill and should not be interpreted as applying to all sites. However, they should prove useful in determining rough approximations of well spacing. To determine vent spacing at a particular site, one should first conduct a series of test drawdowns at various flow rates while measuring head loss as a function of distance from the vent.

Pipe wells are usually constructed of 4- or 6-inch PVC perforated pipe. Other material, such as galvanized iron, may be required if PVC is not compatible with the waste materials. A surrounding layer of gravel pack should be installed to prevent clogging. The pipe vent should be sealed off from the atmosphere with a cement or cement/soil grout so that excess air is not introduced into the system, and methane or volatile toxics cannot leak out. Pipe vents may be installed through a clay cap, as shown in Figure 6-1, to prevent emissions of gases or vapors to the atmosphere. Vent wells may also be installed in a continuous layer of sand overlying the fill, which serves as a permeable channel for the transportation of gases, as shown in drawing (d) of Figure 6-1.

6.1.4 Advantages and Disadvantages

Atmospheric vents are an effective means of control when used in situations where gases freely migrate to a collection point and there is little or no lateral migration. They are not an effective means of controlling lateral migration. Forced ventilation is by far the more effective method for controlling migration. If forced ventilation is used, the flow rate can be increased or decreased as the gas generation or vapor flux rate increases or decreases. This offers a great deal of flexibility of control inherent in the system. At a hazardous waste site where volatile toxics are present, the mass flux rate will decrease with time as the volatiles are dissipated. Thus, ventilation rates can be reduced with time and operating costs will decrease. It is expected that gas vents from forced ventilation are more apt to clog after time, and will need to be replaced. Also, it is expected that more maintenance will be required for forced ventilation than for passive atmospheric vent systems.

6.1.5 Costs

The cost of installing pipe vents is similar to that of groundwater monitoring wells, because the same construction materials are used. Installed costs range from $2.00 to $2.50 per inch diameter per foot of depth for small diameter vent wells, excluding casing costs (U.S. Department of Interior, 1977). Casing costs for PVC range from $4.50 to $6.50 for 4 inch and 6 inch casings, respectively (Leazer Pumps and Wells, 1980). Mushroom tops for ventilation were quoted at $25 each (McCaffray Company, 1980). Costs for elbows to fabricate a u-shaped top can be found in Table 6-5 in Section 6.4.5.

Also, the cost for a small fan applicable to one pipe vent is given in Table 6-5. The maximum cost for a single forced ventilation pipe vent of 4 inches diameter, and 30 feet deep, would then be as follows:

$$\text{Installed cost (max)} = \frac{\$2.50}{\text{in-ft}} \times 4 \text{ in} \times 30 \text{ ft} = \$\ 300$$

$$\text{Cost of casing (max)} = \frac{\$6.50}{\text{ft}} \times 30 \text{ ft} = \$\ 195$$

$$\text{Fan} = \$\ 615$$

$$\text{Total Installed Cost} = \$1{,}110$$

6.2 TRENCH VENTS

6.2.1 General Description

Trench vents are constructed by excavating a deep, narrow trench surrounding the waste site or spanning a section of the area perimeter. The trench is backfilled with gravel, forming a path of least resistance through which gases migrate upward to the atmosphere or to a collection manifold. By diverting flow in this manner, the trench vents form a barrier against lateral migration of methane or toxic vapors. Trench vents are used in combination with liners to form an effective barrier against gas migration. Trenches can be open or capped with clay and fitted with collection laterals and riser pipes venting to the atmosphere or connected to a negative pressure fan or blower. Also, air can be injected into trench vents to form a blanket that controls gaseous migration. Various configurations of trench vents are shown in Figure 6-7.

6.2.2 Application

Trench vents are used primarily to attenuate lateral gas or vapor migration. They are most successfully applied to sites where the depth of gas migration is limited by groundwater or an impervious formation. If the trench can be excavated to this depth, trench vents can offer full containment and control of gases and vapors.

As with pipe vents, the applicability of different trench vent systems depends on whether methane generation is occurring or whether the problem at the site is limited to the control of toxic vapors. Passive open trenches (drawings (a) and (b) in Figure 6-7) may be applicable to the control of toxic vapors in an emergency situation where immediate relief is required. They also can be employed as a permanent control for methane migration; however, their efficiency is expected to be low (Stone, 1978a). An impervious liner can be added to the outside of the trench to increase control efficiency (see Section 6.4). Open trenches are more suitable for sparsely populated areas where they will not be accidentally covered, planted over, or otherwise plugged by outsiders (Bowerman, 1980).

Trench vents may be covered over by clay or other impervious materials and vented either to the atmosphere or to a collection system. Such a system insures adequate ventilation and prevents infiltration of rainfall into the vent. Also, an impervious clay layer can be used as an effective seal against the escape of toxic vapors. Figure 6-3 shows a closed trench with perforated lateral and riser pipes venting to the atmosphere. Drawing (d) in Figure 6-1 depicts a closed trench with pipe vents installed at intervals and connected to a negative pressure system. Another type of forced ventilation in a trench for methane migration control is air injection; in this method, air injected into the trench by a blower forces the gas or vapor back. This system should work well in conjunction with pipe vents installed close to the landfill and inside the circumference of the trench.

6.2.3 Design and Construction

Some important considerations are involved in the proper design and construction of trench vents.

Open vents are subject to infiltration by rainfall runoff and could become clogged by solids. Hence, they should not be located in an area of low relief. It is probably advisable to construct a slope with some of the excavated soil to direct runoff away from the trench as in drawings (a) and (b) of Figure 6-7. Also, if possible, open trenches should be constructed within controlled areas to prevent any safety or vandalism problems.

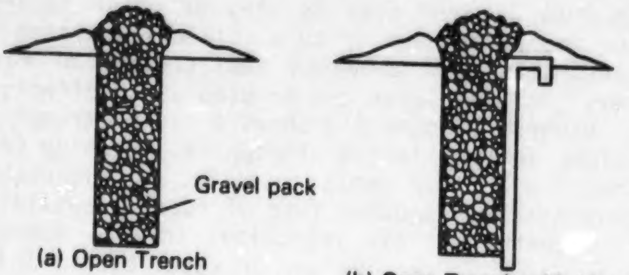
It is important to ensure that the gravel pack in the trench will be permeable enough, relative to the surrounding strata, to transport the gas adequately. Also, in areas of relatively high permeability or wherever safeguards are needed, a liner should be installed on the outside of the trench to prevent bypass (see Section 6.3).

In passive closed trench vents, one can ensure good ventilation by proper design of laterals and risers. One design consisted of 12-inch perforated corrugated lateral pipe with 8-foot corrugated risers spread at 50-foot intervals (Constable et al., 1979).

In closed trench vents with forced ventilation, one can apply the equation and design criteria discussed in Section 6.2.3 for the control of methane and toxic vapors, since gaseous diffusion is omnidirectional. Acreages should be converted to smaller areal units, which can be applied to the face area at

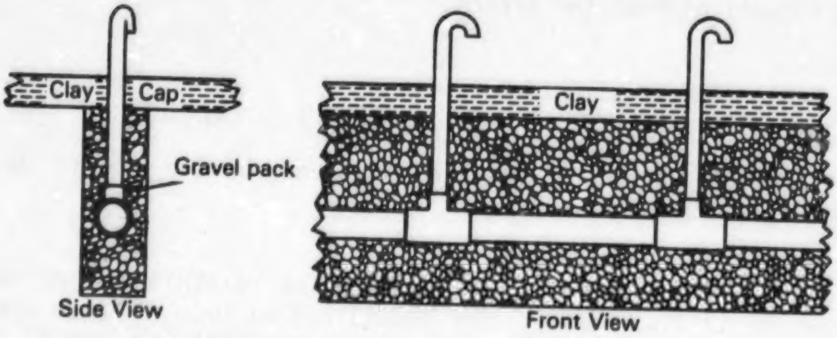
FIGURE 6-7

DESIGN CONFIGURATIONS OF TRENCH VENTS

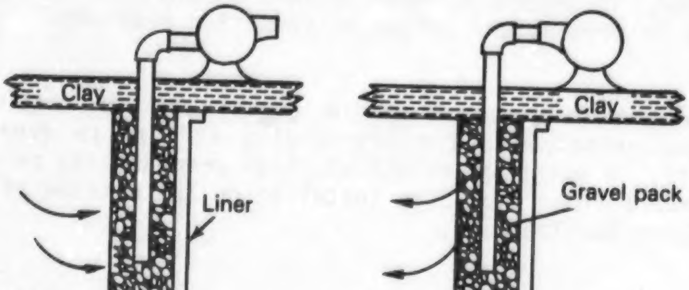

Gravel pack

(a) Open Trench

(b) Open Trench with Liner


Clay
Cap
Gravel pack
Side View
Clay
Front View

(c) Closed Trench with Lateral and Risers


Clay
Liner
Clay
Gravel pack

(d) Induced Draft

(e) Air Injection

the trench. Pipes for trench ventilation can probably be placed at greater distances, since the trench fill is composed of very permeable material. If a liner is used, the pipe vents can probably be placed farther apart, since the normal radial influence of the vents will be channeled even farther along the trench.

It is important that trenches extend to form a continuous impermeable seal with the groundwater or an unfractured impervious stratum. If the trench is not bottom-sealed in this way, gases may migrate under, and completely bypass, the trench.

6.2.4 Advantages and Disadvantages

Trench vents with passive ventilation have not been found to offer very effective control of migrating gases. Passive trenches with an impermeable liner may offer the required degree of effectiveness; however, the installation of a liner will generally be economical only if the required depth is 10 feet or less (Stone, 1978b). Also, since a liner is subject to tearing or cracking, a substantial risk of failure is involved, so passive trench vents with liners are not recommended if the methane problem is substantial. Also, trench vents may become plugged by soil particles with time, thereby reducing their long-term effectiveness.

Induced draft systems are by far the most effective and controllable technology to remedy gas migration problems at waste sites. As they are more easily maintainable than trench vents with liners, their long-term effectiveness is expected to be greater.

6.2.5 Costs

The costs of installing trench vents include those for trench excavation and dewatering, gravel backfill and liners, laterals, and risers, if applicable. Unit costs for most of these items are given in Table 6-1. A trench vent 20 feet deep, 4 feet wide, and 500 feet long with laterals, risers, and a synthetic liner is costed as follows:

- The total volume of the trench is:

  $(500 \text{ ft})(4 \text{ ft})(20 \text{ ft}) = 40{,}000 \text{ ft}^3$ or 1500 yd^3

  The surface area of the side of the trench is:

  $(500 \text{ ft})(20 \text{ ft}) = 10{,}000 \text{ ft}^2$

TABLE 6-1

UNIT COSTS FOR INSTALLATION OF TRENCH UNITS

	Assumptions	Costs
Trench excavation	20'deep, 4' wide, by backhoe	$1/cubic yard[1]
Spread excavated material	Spread nearby and grade and cover trench	$.66/cubic yard[1]
Wellpoint dewatering	500' header, 8" diameter, for one month	$75/linear foot[1]
Gravel	Buy and haul from pit 2 miles, backfill with dozer	$7.60/cubic yard[1]
Sheet piling	Pull and salvage	$5.70/square foot[1]
Walers, connections, struts	2/3 salvage	$105/ton[2]
Lateral with risers	12" corrugated polyethylene lateral, 6" PVC risers, 15' long every 50'. 500' lateral	$6.50/linear foot[1]
Liner	Hypalon (36 mil) Bracketed with heavy-weight geotextile fabric	$2.10 - $2.65/ square foot[4]
	4" gunite layer with mesh	$4.62 - $8.40/ square foot[1]

---

[1] Godfrey, 1979; Costs are total including contractor overhead and profit.
[2] Godfrey, 1979; Materials only.
[3] Camwell Corp., 1980; Assumed 50% installation cost.
[4] Universal Linings, 1980.

- The total cost of trench excavation is as follows:
  - Trench excavation cost

    ($1/cubic yard)(1500 cubic yards) = $1500
  - Spreading cost

    ($0.66/cubic yard)(1500 cubic yards) = $990
  - Well point dewatering cost:

    Assume installation time of one month

    ($75/linear feet)(500 feet) = $37,500
  - Sheet piling cost.

As a rule-of-thumb in the construction business, the total area of sheet piling needed can be estimated by multiplying the side (2) areas of trench by a factor of 1.6 to account for the allowance area for the interlocking devices (Staples, 1980). Thus the total area of sheet piling required is:

$$(2)(10{,}000\ \text{ft}^2)(1.6) = 32{,}000\ \text{ft}^2.$$

According to ARMCO, the average weight of sheet piling per square foot of wall is 10 lbs. (ARMCO, 1980). Thus the total tonnage of the sheet piling can be calculated as follows:

$$(32{,}000\ \text{ft}^2)(10\ \text{lbs/ft}^2) = 320{,}000\ \text{lbs or } 145\ \text{tons}.$$

The cost of the sheet piling can then be calculated as follows:

$$(32{,}000\ \text{ft}^2)(\$5.70/\text{ft}^2) = \$182{,}500$$

- Waler, connection, and strut costs:

  As a rule-of-thumb in the construction business, the total tonnage of walers, connections and struts are 20 percent of the weight of sheet piling (Staples, 1980). Therefore, the tonnage required for walers and struts is:

  (145 tons)(0.2) = 29 tons.

Therefore, the cost for walers and struts is:

($105/ton)(29 ton) = $3,050

Therefore, the total cost of trench excavation is:

$1500 + $990 + $37,500 + $182,500 + $3,050 = $225,540

In addition to trench excavation costs, costs for the gravel fill, the lateral and rise pipe, and the liner must be added:

- Cost of gravel

  (1500 cubic yards)($7.60/cubic yard) = $11,400

- Cost of laterals and risers

  (500 feet)($6.50/linear feet) = $3,250

- Cost of liner (average)

  - Hypalon liner bracket with geotextile fabric
    Total area = wall area of trench, plus additional area needed for anchoring.

    Wall area of trench = 10,000 ft^2

    Additional area = (4 ft. x 4 ft) x 500 ft = 4,000 ft^2

    Total area = 14,000 ft^2

  Cost = (14,000 ft^2)($2.38/$ft^2$) = $33,320

The total cost of the trench vent described above would then be as follows:

$225,540 + $11,400 + $3,250 + $33,320 = $273,510

It is obvious that costs for an effective trench vent system can be quite substantial.

6.3 GAS BARRIERS

6.3.1 General Description

Barriers against the migration of gases and vapors are employed in a number of ways at waste disposal sites, usually in conjunction with other remedial measures. An effective barrier against gas flow must consist of a material with low gas permeability. Materials found to prevent gas migration include compacted clay, concrete slurry walls, gunite, and synthetic liners.

6.3.2 Application

Compacted clay used as a cap to prevent infiltration into waste disposal sites has also been found to inhibit the vertical migration of methane gas

(Stone, 1978a). This material can then be purposely used for vertical gas migration control; however, it does not form an ideal barrier because it may crack upon drying (Thibodeaux, 1979). It may be most applicable for containing methane prior to gas recovery rather than as a remedial measure, since more stringent control is desirable. Compacted clay is not recommended for the control of vertical migration of toxic vapors, where a highly impermeable barrier is required. Effectiveness of a clay barrier for gas and vapor control may be improved by using it in conjunction with a permeable sand sublayer and forced ventilation, as shown earlier in Figure 6-1(d). Also, a cover of topsoil and vegetation may serve to trap moisture and prevent clay layers from drying out.

Other remedial actions not intended for gas control may incidentally form effective gas barriers. Grout curtains and concrete slurry walls, which are used for groundwater containment, fall under this category. However, no data were available on gas permeability of these materials, so their potential effectiveness as gas barriers could not be determined.

Synthetic liners also have been used to prevent the migration of gases and vapors. As described earlier, they can be used vertically in combination with trench vents to form an effective barrier (see Section 6.3). They also can be installed as a top cover the landfill for vertical gas control as well as infiltration control (see Section 3.1). When installed in trench vents, liners must extend to form a continuous bottom seal, either with groundwater or impermeable bedrock. Their applicability is limited to areas where the depth to groundwater or bedrock is less than the maximum digging depth of a backhoe, approximately 50 feet (with clamshell attachment).

Synthetic liners used alone to prevent gas migration have not performed well (Stone, 1978b). Gunite has also been used to form a vertical gas barrier (Bowerman, 1980). However, its use is restricted to areas where it can be applied easily, such as in trenches less than 10 feet deep.

6.3.3 Design and Construction Considerations

Two important considerations in constructing gas barriers are selection of materials impermeable to gases, and maintenance of barrier integrity during installation. Two synthetic materials exhibit low permeability to gases: Hypalon, a chlorosulfonated polyethylene; and Neoprene, a well-known synthetic elastomer. Hypalon's moderate cost and high seam strength make it more desirable than high-cost neoprene liners, which must be bound with epoxy and which form a seal of lower strength (Universal Linings, 1980). Usually a reinforced liner is recommended, in which one or more layers of nylon or dacron fiber scrim are added to increase strength.

To maintain liner integrity during installation in trench vents, the liner must be protected against perforation by stones in the earth wall and by the gravel fill. For this reason, it is recommended that the liner be sandwiched between two layers of heavy weight jute backing, known as geotextile.

When installing liners to control toxic vapors, consideration must be given to the liners' compatibility with the vapors it may contact. Table 6-2 gives the chemical resistances of Hypalon and Neoprene with various gases and vapors. It can be seen that many toxic organic chemicals may be incompatible with these synthetic liners, and other materials may be required for toxic vapor control.

One possibility for organic vapor control is the installation of a Teflon-coated fiberglass liner. Such a liner would withstand attack from almost all chemicals. However, available liner material is only 10 mil thick, which may prove too light for this application. Also, this type of liner has never been tested in the field, and since it is available only in 38-inch-wide sheets, field bonding may present some problems. Finally, this liner is very expensive.

Liners must be properly anchored when installed. Figure 6-8 shows a recommended anchoring technique (B. F. Goodrich), a minimum of 3 1/2 feet of border is required for proper liner anchoring in this technique.

Gunite, a form of sprayed concrete and admixed materials, such as asphaltic concrete, may also be suitable for the control of organic vapors. However, they have higher permeability than the synthetics (if compatible), and tend to crack under conditions of differential settlement and weathering (JRB; EMCOM, 1980).

6.3.4 Advantages and Disadvantages

Gas barriers afford fair containment of gases and vapors from landfills, but must be used in conjunction with other gas control technologies to be effective. Hypalon synthetic liners are good barriers for use in trench vents, but are not compatible with many organic vapors. Gunite and asphaltic concrete may be successful liner materials; however, they are subject to cracking because of differential settlement and weathering. The cost of installation of gunite seems prohibitive. For control of vertical migration, compacted clay offers an economical and effective seal but will crack unless kept moist. Clay may be an effective barrier for use in combination with trench vents. Teflon liners, although impermeable and resistant to almost everything, are expensive and have not been field-tested.

TABLE 6-2

CHEMICAL RESISTANCE CHART (DUPONT)

Vapor type	Hypalon*	Neoprene*
Acetic acid (glacial)	X	C
Acetone	T	B
Benzene	C	C
Butane	A	A
Butyraldehyde	X	C
Carbon tetrachloride	C	C
Cyclohexane	C	C
Dioctyl phthalate	C	C
Ethyl acetate	C	C
Ethyl alcohol	A (158°F)	A (158°F)
Formic acid	A	A
Gasoline	B	B
Hydrochloric acid (Conc.)	-	-
Hydrocyanic acid	B	A
Hydrogen sulfide	B	A
Kerosene	X	B
Methyl alcohol	A (158°F)	A (158°F)
Methylene chloride	C	C (100°F)
Methyl Ethyl ketone	C	C
Naphtha	B	C
Perchloroethylene	X	C
Toluene	C	C
Trichloroethylene	C	C
Xylene	C	C

*Cured Sheet

A - Chemical has little or no effect.
B - Chemical has minor to moderate effect.
C - Chemical has severe effect.
X - No data - not likely to be compatible.

Unless otherwise noted, concentrations of aqueous solutions are saturated.
All ratings are at room temperature, unless specified.

FIGURE 6-8

DESIGN OF ANCHOR TRENCH

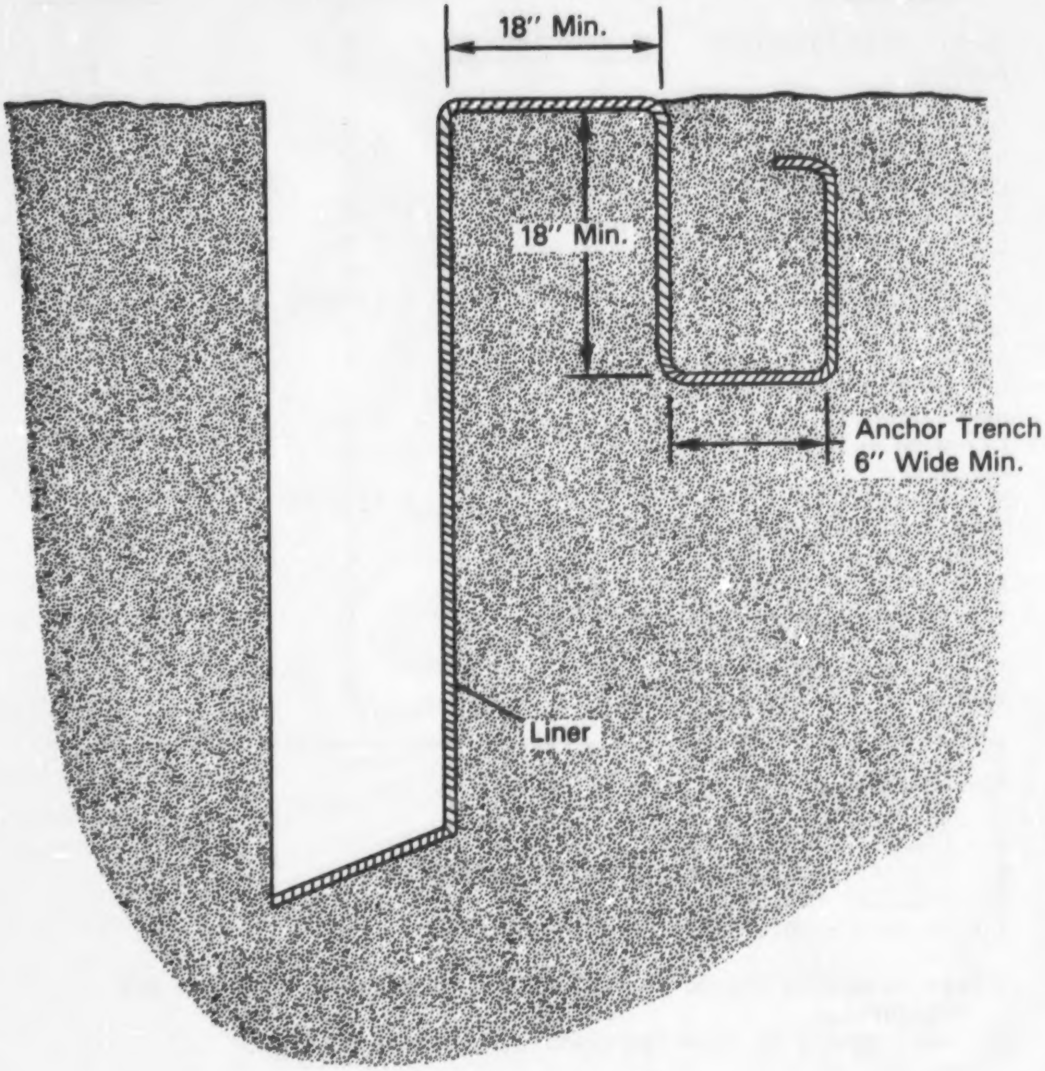

18" Min.
18" Min.
Anchor Trench
6" Wide Min.
Liner

TABLE 6-3

COSTS FOR GAS BARRIERS

	Assumptions	Total costs	Information source(s)
Synthetic Liners			
Hypalon	36 mil thickness	\$0.60 - 0.65/ft^2	1
Teflon	10 mil thickness	\$2.22/ft^2	1,2
Geotextile backing	Heavyweight, two layers	\$1.50 - 2.00/ft^2	1
Gunite	4" layer with wire mesh	\$4.62 - \$8.40/ft^2	3
Asphaltic concrete	4" - 6" layer, including base	\$0.33 - 0.56/ft^2	4
Clay	Material cost, hauling, backfill by dozer, vibratory compaction every 6". For trench vent only.[5]	\$0.43/ft^3	3,4

---

[1] Information provided by Universal Linings, Inc.
[2] Approximate
[3] Godfrey, 1979
[4] Lutton, et al., 1979
[5] Cost for clay surface liner given in Table 3-2

6.3.5 Costs

The costs for installing different types of liners include those for materials and installation in the field. For trench vents, two layers of geotextile backing should be used to protect synthetic liners from holes caused by rocks and other protuberances. When determining cost estimates for liners, at least 3 1/2 feet should be added to liner width to allow for proper surface anchoring. Also, the area required to line the bottom of the trench should be added to the total area of the liner. Determining liner installation costs is addressed in Section 6.2.5.

6.4 GAS COLLECTION SYSTEMS

6.4.1 General Description

As discussed earlier, gas vents are frequently connected to a fan or blower, or to a manifold (with other vents) with a centralized fan or blower. The fan may then discharge into a gas treatment system, such as a flare or carbon unit, or into a gas recovery unit. The section between the vent(s) and the gas treatment or recovery system has been termed the "collection" system. The components of a collection system include piping (ductwork), fittings, fans (or blowers), and any flow adjustment or measuring device located between the vent(s) and the treatment or recovery system.

6.4.2 Application

Gas collection systems are employed whenever forced ventilation of pipe or trench vents is designated. In their simplest form, they may consist of a single pipe vent connected to a fan and directly discharging to the atmosphere or to a treatment device. Such a simple system may be used at a small disposal area. Since the radius of influence of a single forced ventilation pipe has been found to be as high as 200 to 300 feet, it is conceivable that a simple vent and fan system could be used on areas as large as 5 to 6 acres (Carlson, 1977). A simple vent and fan collection system may also be applicable to areas where different wastes have been partitioned, and where it is undesirable to mix vented gases or vapors, either because of incompatibility or because of separate treatment requirements. Also, separate fans may be installed on individual pipe vents in the event of an emergency situation requiring immediate action; fans can be installed more quickly than a manifold collection system.

The more commonly specified collection system will consist of a manifold system in which several pipe vents connect to an exhaust header, which in turn may be one of several header branches leading into a larger manifold and ultimately to a large centrifugal fan. Each branch take-off may be supplied with a butterfly valve for balancing, and a flow meter for checking flow rate. These more complicated collection systems are applicable to larger disposal areas (greater than 5 acres) and in situations where methane is to be collected for burnoff in a centralized flare or recovered as a fuel gas, or where toxics are collected for treatment, as in a well-controlled centralized treatment facility. Gas collection systems are often underground, with access ports built to valves, moisture traps, and other frequently adjusted features.

6.4.3 Design and Construction Considerations

The primary consideration in designing a collection system for a single pipe vent is the proper selection of a fan. As a rough guide in determining static pressure and flow rates required for disposal site ventilation, Figures 6-5 and 6-6 can be used. As mentioned earlier, a flow rate of 50 ft^3/min and a static pressure of 3 inches of water were found to draw gas at a radius of 130 feet in compacted refuse (Carlson, 1977). A small-diameter blower is required to pull 3 inches of pressure at such low flow rates; however, the drive unit may require a substantial power rating, due to inefficiency of such units. One unit found to be applicable had a drive rated at 1/3 horsepower (Britton, 1980). Since the blower is outside, it should be fully enclosed. If explosive gases are expected to be present, an explosive-proof motor housing may be specified. Also, coating the internal blades is required if corrosive materials, such as hydrogen sulfide, may be present. For proper selection of a fan, one must use manufacturer's design charts as shown in Figure 6-9. The flow rate moving through the fan is charted against the total static pressure of the fan. This static pressure can be related to the static pressure at the fan inlet by the following equation (Trane Company, 1965):

$$\text{Fan } P_s = P_{t(o)} - P_{t(i)} - P_{v(o)}$$

where P_s = Static pressure of fan, inches of water
$P_{t(o)}$ = Total pressure of outlet, inches of water
$P_{t(i)}$ = Total pressure at inlet, inches of water
$P_{v(o)}$ = Velocity pressure at outlet, inches of water
P_t = Static pressure + velocity pressure

Velocity pressure can be calculated by the equation:

$$P_v = \frac{dV^2}{1.2 \times 10^6}$$

where V = Velocity feet/min
d = Gas density, lb/ft^3
P_v = Velocity pressure, inches of water

The velocity at the fan outlet is found on fan charts as shown in Figure 6-9. In the low velocities found in landfill ventilation systems, P_V is negligible and total inlet pressure is essentially all static pressure. In situations where the fan is discharging to the atmosphere, static pressure (gage) of the outlet would be equal to zero.

Besides fan selection, the design of manifold collection systems involves duct sizing and providing for flow rating and flow balancing in the system. Normally, noise level is the chief consideration in selection of the design flow rate near the fan. In a disposal site ventilation system, the design flow rate near the fan will be specified by the sum of the flow rates of each vent in the system. The flow rate per vent, along with the required static pressure for ventilation, must be determined by preliminary ventilation testing in the area, as described earlier. Because a manifold is involved, the friction loss in the piping is an important consideration; the designer must select pipe diameters that enable the fan to handle both the static pressure required at the vent, and the head loss from friction of the piping system.

Friction loss is expressed in inches of water per length of pipe; it is directly related to flow rate and inversely related to pipe diameter. In other words, as pipe diameter increases and flow rate decreases, the head loss due to friction decreases, and vice versa. Figure 6-10 provides a method of solving frictional loss ("pressure drop" in the figure) if one knows the flow rate, the ambient temperature, the molecular weight of the gas, and the pipe diameter. Other charts are available; however, they do not take into consideration the ambient temperature, which is important in an outside system.

Besides the friction loss from flow through the pipe, an additional frictional head loss is incurred whenever the flow is disturbed, such as when an elbow, tee, valve, constriction, enlargement, or anything other than the normal pipe wall is encountered. In many cases, the frictional loss from these fittings can be related to an equivalent length of pipe. For gases similar to air, the following equation can be used to approximate an equivalent pipe length, where K is a constant depending on the type of fitting. Values for K can be found in Table 6-4.

$$\frac{Le}{D} = 55K$$

Where Le = equivalent length, inches
D = inside pipe diameter, inches

Figure 6-11 provides a direct means of determining head loss (in inches of water) for elbows and branch connections.

FIGURE 6-9

FAN DESIGN CHART

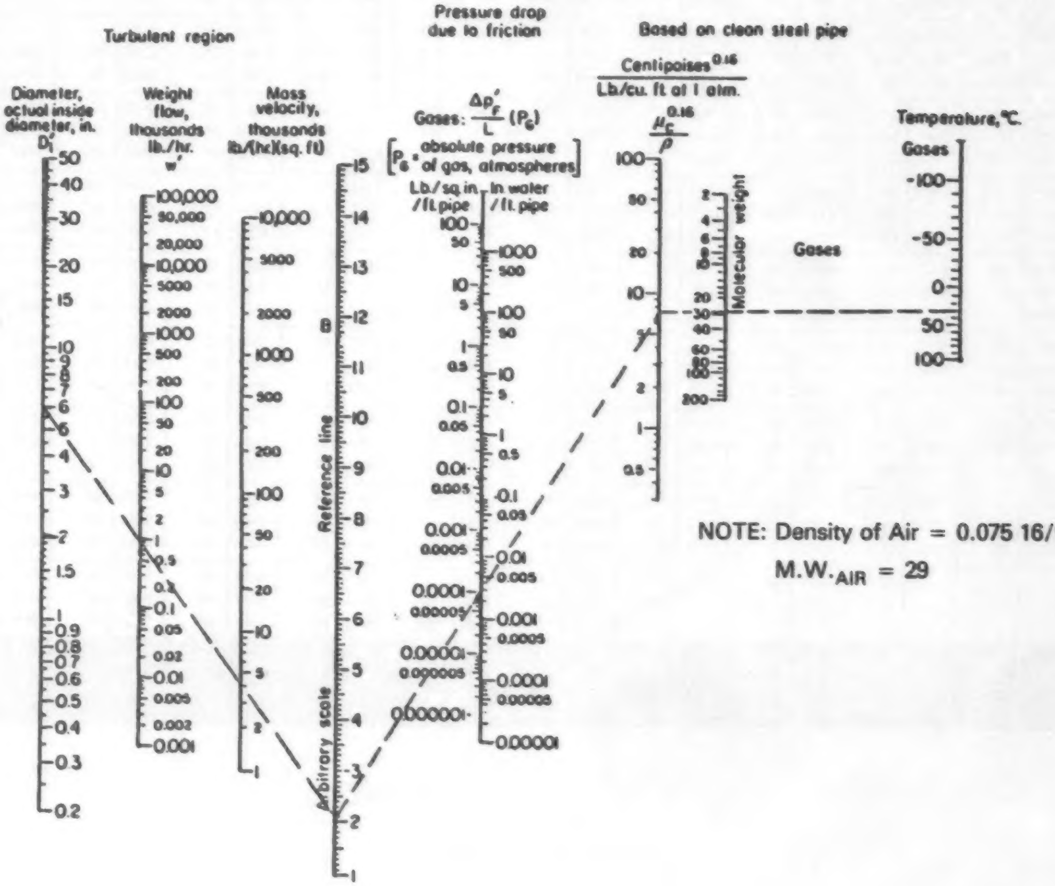
(Source: Aladdin, 1980)

CFM	OV	1/4" SP		3/8" SP		1/2" SP		5/8" SP		3/4" SP		1" SP		1¼" SP		1½" SP		1¾" SP		2" SP	
		RPM	BHP	RPM	BHP	RPM	BHP	RPM	BHP	RPM	BHP	RPM	BHP	RPM	BHP	RPM	BHP	RPM	BHP	RPM	BHP
515	600	845	.03	966	.05	1083	.07	1196	.08	1300	.11										
601	700	912	.04	1015	.06	1118	.07	1219	.09	1318	.12	1503	.16								
687	800	987	.05	1077	.07	1168	.08	1258	.11	1347	.13	1520	.18	1683	.23	1831	.29				
773	900	1068	.06	1148	.08	1228	.10	1309	.12	1389	.14	1547	.19	1701	.25	1847	.31	1984	.38	2109	.45
859	1000	1153	.08	1225	.09	1297	.11	1369	.14	1442	.16	1586	.21	1728	.27	1866	.33	2000	.40	2127	.47
945	1100	1241	.09	1306	.11	1371	.13	1437	.16	1503	.18	1634	.23	1765	.29	1894	.35	2021	.42	2144	.49
1031	1200	1331	.11	1391	.13	1450	.16	1510	.18	1571	.20	1691	.26	1812	.32	1932	.38	2060	.45	2167	.52
1117	1300	1423	.13	1478	.16	1533	.18	1588	.21	1644	.23	1755	.29	1866	.34	1978	.41	2088	.48	2198	.55
1203	1400	1516	.16	1567	.18	1618	.21	1669	.24	1721	.26	1824	.32	1927	.38	2031	.44	2134	.52	2237	.59
1289	1500	1611	.19	1658	.21	1706	.24	1753	.27	1801	.30	1897	.36	1994	.42	2090	.48	2187	.56	2283	.63
1375	1600	1706	.22	1750	.25	1795	.28	1839	.31	1884	.34	1974	.40	2064	.46	2155	.53	2245	.60	2336	.68
1461	1700	1802	.26	1843	.29	1885	.32	1927	.35	1969	.38	2054	.44	2139	.51	2224	.58	2309	.65	2394	.73
1547	1800	1898	.30	1938	.33	1977	.36	2017	.39	2056	.42	2136	.49	2216	.56	2296	.63	2376	.71	2457	.78
1633	1900	1995	.34	2033	.38	2070	.41	2107	.44	2145	.48	2220	.55	2296	.62	2372	.69	2448	.77	2524	.85
1719	2000	2093	.39	2128	.43	2163	.46	2199	.50	2235	.53	2306	.60	2378	.68	2450	.76	2522	.84	2594	.92
1805	2100	2190	.45	2224	.48	2258	.52	2292	.56	2325	.59	2393	.67	2462	.75	2530	.83	2599	.91	2667	.99
1891	2200	2289	.51	2321	.55	2353	.58	2385	.62	2417	.66	2482	.74	2547	.82	2612	.90	2678	.99	2743	1.07
1977	2300	2387	.57	2418	.61	2448	.65	2479	.69	2510	.73	2572	.81	2634	.89	2696	.98	2759	1.07	2821	1.16
2063	2400	2486	.64	2515	.69	2544	.73	2574	.77	2603	.81	2663	.89	2722	.98	2782	1.07	2841	1.16	2901	1.25
2149	2500	2584	.71	2613	.76	2641	.81	2669	.85	2697	.89	2754	.98	2811	1.07	2868	1.16	2926	1.25	2983	1.35
2235	2600	2683	.79	2710	.85	2738	.89	2765	.94	2792	.98	2847	1.07	2901	1.16	2956	1.26	3011	1.35	3066	1.45
2321	2700	2782	.88	2809	.94	2835	.99	2861	1.03	2887	1.08	2940	1.17	2992	1.26	3045	1.36	3098	1.46	3151	1.56
2407	2800	2882	.97	2907	1.03	2932	1.08	2957	1.13	2983	1.18	3033	1.28	3084	1.37	3135	1.47	3186	1.57	3237	1.68
2493	2900			3005	1.13	3030	1.19	3054	1.24	3078	1.29	3127	1.39	3176	1.49	3225	1.58	3274	1.69	3324	1.80
2579	3000			3104	1.24	3128	1.30	3151	1.36	3175	1.41	3222	1.51	3269	1.61	3317	1.72	3364	1.82	3412	1.93
2751	3200			3302	1.47	3324	1.54	3346	1.61	3368	1.44	3412	1.77	3457	1.88	3801	1.99	3545	2.10	3590	2.21

FIGURE 6-10

NOMOGRAPH FOR SOLUTION OF FRICTION LOSSES IN PIPE

(Source: Perry, 1973)
See Copyright Notice, Page 496.



NOTE: Density of Air = 0.075 16/ft^3

M.W.$_{AIR}$ = 29

TABLE 6-4

K VALUES FOR FITTINGS AND VALVES

Type of fitting or valve	Additional friction loss, Equivalent number of velocity heads, K
45° ell, standard	0.35
45° ell, long radius	0.2
90° ell, standard	0.75
Long radius	0.45
Square or mite	1.3
180° bend, close return	1.5
Tee, standard, along run, branch blanked on	0.4
Used as ell, entering run	1.0
Used as ell, entering branch	1.0
Branching bow	1
Coupling	0.04
Union	0.04
Gate valve, open	0.17
3/4 open	0.9
1/2 open	4.5
1/4 open	24.0
Diaphragm valve, open	2.3
3/4 open	2.6
1/2 open	4.3
1/4 open	21.0
Globe valve, bevel seat, open	6.0
1/2 open	9.5
Composition seat, open	6.0
1/2 open	3.5
Plug disk, open	9.0
3/4 open	13.0
1/2 open	36.0
1/4 open	112.0
Angle valve, open	2.0
Y or blowod valve, open	3.0
Plug cock = 5°	0.05
10°	0.29
20°	1.36
40°	17.3
60°	118.0
Check valve, swing	2.0
Disk	10.0
Bail	70.0
Foot valve	15.0

--continued--

TABLE 6-4

K VALUES FOR FITTINGS AND VALVES

Type of fitting or valve	Additional friction loss, Equivalent number of velocity heads, K
45° ell, standard	0.35
45° ell, long radius	0.2
90° ell, standard	0.75
Long radius	0.45
Square or mite	1.3
180° bend, close return	1.5
Tee, standard, along run, branch blanked on	0.4
Used as ell, entering run	1.0
Used as ell, entering branch	1.0
Branching bow	1
Coupling	0.04
Union	0.04
Gate valve, open	0.17
3/4 open	0.9
1/2 open	4.5
1/4 open	24.0
Diaphragm valve, open	2.3
3/4 open	2.6
1/2 open	4.3
1/4 open	21.0
Globe valve, bevel seat, open	6.0
1/2 open	9.5
Composition seat, open	6.0
1/2 open	3.5
Plug disk, open	9.0
3/4 open	13.0
1/2 open	36.0
1/4 open	112.0
Angle valve, open	2.0
Y or blowod valve, open	3.0
Plug cock = 5°	0.05
10°	0.29
20°	1.36
40°	17.3
60°	118.0
Check valve, swing	2.0
Disk	10.0
Bail	70.0
Foot valve	15.0

--continued--

TABLE 6-4 (Continued)

Type of fitting or valve	Additional friction loss, equivalent number of velocity heads, K
Water meter, disk	7.0
Piston	15.0
Rotary (star-shaped disk)	10.0
Turbine wheel	5.0

Source: Perry, 1973
See Copyright Notice, Page 496.

When piping is sized, the total head loss must be compatible with the selection of a fan. For example, if the static pressure required at each vent is 3 inches of water, and a fan is found to pull the required flow rate at a static pressure of 6 inches of water, then 3 inches of head can be lost as friction along the length of the manifold. Incidentally, it should be possible to vary the static pressure of the fan and still be in proximity of the maximum efficiency line, as shown in Figure 6-9. The longest fan-to-vent distance in the system should be used to determine the allowable friction loss per unit vent, keeping in mind that take-offs and other fittings will contribute somewhat to the total head loss.

Once the unit friction loss has been specified, the designer or cost estimator can work backwards from the fan to the first branch, and then to succeeding branches, using the unit friction losses and calculated flow rates to determine pipe diameters (from Figure 6-10).

The following simplified design problem illustrates the calculation method. Assume a collection system must be designed in which a fan is connected to four pipe vents. Previous testing indicated that a flow rate of 50 ft^3/min at 3 inches of static head would have a radius of influence of 130 feet. For sufficient overlap, it was determined that the vent spacing should be 200 feet. Therefore, a total of 600 feet was required between the first vent and the fan. The system is pictured in Figure 6-12. An additional 20 feet was added from the last vent to the fan for a total maximum distance of 620 feet.

FIGURE 6-11

ELBOW AND BRANCH TAKE-OFF LOSSES

(Source: Trane, 1965)

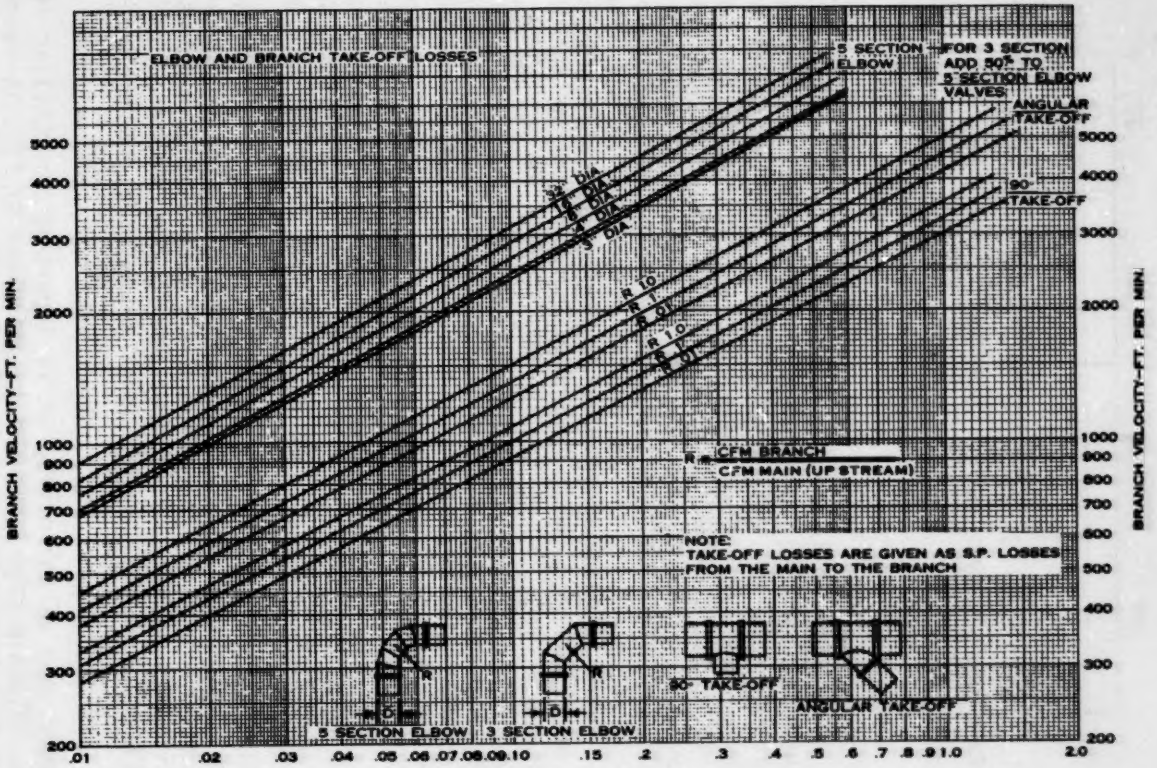
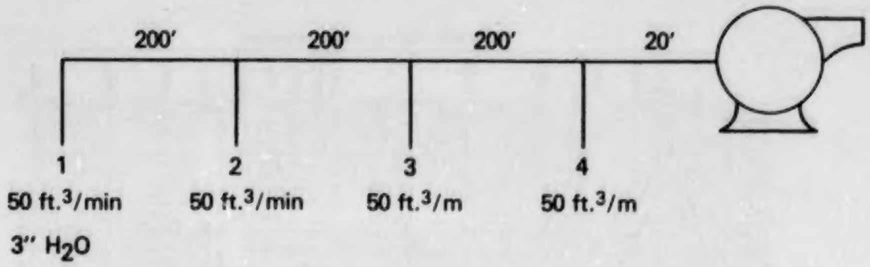


FIGURE 6-12

EXAMPLE OF COLLECTION SYSTEM



The fan size must handle 200 ft^3/min at a static pressure of 3 inches of water, plus the head loss. From Figure 6-9, a size 9 fan can handle about 218 ft^3/min at about 6" SP with a power requirement of about 0.36 HP. If we assume that 1 inch of static pressure is required at the fan discharge (to push gas through a treatment system), then 2 inches of water is allowable for head loss. Using the maximum length of pipe, the design friction loss is 0.003 inches of water per foot of pipe. If the ambient temperature is 30°C, and the molecular weight of the gas is assumed to be approximately equal to air, then Figure 6-10 can be used to determine appropriate pipe diameters as a function of flow rate. At the section between the fan and the fourth vent, there will be a total flow rate of 200 ft^3/min or 900 lb/hr, if the gas is assumed to be air. Thus, from Figure 6-10, a 6-inch pipe would be required. In the next section, 150 ft^3/min, or 675 lb/hr, would be flowing, and the recommended pipe diameter also would be about 6 inches. In the next section, 450 lb/hr would be flowing, and a 5-inch pipe would be required. Finally, in the last section, 225 lb/hr of gas would be flowing, and about a 4-inch pipe would be required. It is important to be conservative and always scale up to the larger pipe size when between sizes. The above example does not include friction loss from take-offs and fittings, so in the actual design, the pipe diameters may be slightly larger, or it may be more economical to have the fan operate faster, and at a higher power requirement, to pull more static pressure. The system will have to be balanced, since the design basis is uniform fraction drop rather than insurance of the same static pressure at each branch take-off (SMACNA, 1967). Balancing the system can be accomplished with the use of butterfly valves in all vents except the one farthest from the fan.

As gases may be corrosive, it is important to consider the construction materials. Plastic pipe, such as polyethylene or PVC, is recommended for use with gases that may contain inorganic corrosives such as hydrogen sulfide or hydrochloric acid vapors. Galvanized iron may be more suitable for organics, since plastic pipe may not be compatible with certain organic chemicals. Another alternative is the use of asbestos cement-bonded pipe, or ABS pipe. Also, it may be desirable to design the collection system underground. In this case, such design considerations as corrosion protection, compressive strength of material, and access for system adjustments and monitoring become important.

6.4.4 Advantages and Disadvantages

The major advantages of a simple single-fan/vent collection system are its low cost and low maintenance requirement. Its major disadvantage is that it can be applied only to sites smaller than 5 or 6 acres. Manifolded collection systems are more costly, more complex in design and construction, and require more maintenance, such as replacement of balancing valves and periodic rechecks of balancing.

6.4.5 Costs

Some unit costs have been developed from a consideration of typical construction materials and sizes found in other landfill ventilation systems. They are presented in Table 6-5.

The previous above-ground example collection system, as shown in Figure 6-12, has been costed and calculations follow:

- Cost of Fan

  Fan size must handle 200 ft^3/min at a static head of 6 inches. The second fan listed in Table 6-5 is applicable. Capital and operating costs can be assumed to be proportional to the flow rate. Therefore, the following costs have been calculated from the given cost ranges:

  Total installed cost = $1410
  Annual operating cost = $ 20

- Pipe and Fittings

  As shown in Figure 6-12, about 620 feet of pipe will be required for a PVC collection system for four pipe vents. In addition, four tees will be required to connect the vents. Also, four balancing valves will be required in the system. To simplify calculations, it will be assumed that all manifold components are sized for 6-inch piping diameters:

  - Pipe
    (620 feet) ($17.15/foot) = $10,630
  - Tees
    (4) ($52.12/each) = $ 210
  - Butterfly valves
    (4)($240/each) = $ 960

TABLE 6-5

UNIT COSTS OF COMPONENTS FOR GAS COLLECTION SYSTEMS

Fans Flow rate (cfm)	Total installed cost ($)	Annual operating cost ($)
0-136 @ 3" H_2O	615[2]	15[4]
135-600 @ 8" H_2O	1400-1465[3]	11-69
500-2000 @ 8" H_2O	1900-2050[3]	11-230
1900-6000 @ 8" H_2O	4175=4665[3]	34-690
Manhole	490[5]	

Total installed costs ($/ft)

	4"	6"	8"	10"
Pipe[5]				
PVC[5]	12.35	17.15	-	-
Asbestos Bonded[6]	1.33	1.78	2.73	4.16
Galvanized Iron[5]	17.90	35.00	46.00	61.00
Elbows				
PVC[7]	19.85	41.90	73.50	146.00
Galvanized iron[5]	31.00	70.70	130.40	190.30
ABS[5,6]	15.50	24.86	31.50	75.87
Tees				
PVC[7]	28.00	52.20	106.70	214.00
Galvanized iron[5]	48.80	111.30	217.70	366.70
ABS[5,6]	15.50	24.86	34.50	75.70
Butterfly valves				
Cast Iron[5]	265.00	390.00	590.00	845.00
PVC[8]	145.00	240.00	380.00	500.00
Flow meters[5]	810.00	1020.00	1175.00	-

---

[1] Belt-driven, utility mount, weather cover, and corrosion-resistant coating

[2] Britton Company, 1980

[3] McCaffray Company, 1980

[4] Cost = Fan Brake HP x 0.746 $\frac{\text{KW}}{\text{HP}} \times \frac{8{,}760\ \text{hr}}{\text{yr}} \times \frac{\$0.007}{\text{KW-hr}}$

[5] Godfrey, 1979

[6] McMahon and Pereira, 1979

[7] Camwell Corp., 1980

[8] Plastic Piping Systems, 1980

- Flow meter
  (1) ($1020) = $ 1,020

The total installed cost for the collection system would be as follows:

Total installed cost = $1410 + $10,630 + $210 +
$960 + $1020 = $14,230

Annual operating costs are made up of power costs, as previously discussed, plus maintenance costs. It is assumed that costs for maintenance are 4 percent of total installed costs, therefore:

Maintenance costs = (0.4) ($14,230) = $570
Total annual operating costs = $570 + $20 = $590

6.5 GAS TREATMENT SYSTEMS

6.5.1 General Description

Gases from waste disposal sites frequently contain malodorous and toxic substances, and thus require treatment before release to the atmosphere.

Several basic types of gas treatment are applicable: adsorption by carbon; thermal oxidation; and ranking. Carbon adsorption systems are either non-regenerative or regenerative. Thermal oxidation systems include the use of a flare or afterburner, depending on the desired control requirements. Tankers have been used successfully to remove specific types of constituents, especially as required for gas recovery systems (see next section).

Carbon adsorption systems are composed of a drum, tank, or other container that supports a bed of activated carbon. Contaminated gases flow through the carbon and are adsorbed on the carbon surface due to Van der Waals attraction and chemical bonding. The adsorbate can be removed or desorbed from the carbon by raising the temperature, and the carbon can then be regenerated. In the actual process, steam is used as the heat source for the desorption process. The activated carbon can be either regenerated on-site, by a regeneration system attached to the process, or it can be removed from the contaminant and be regenerated off-site.

Thermal oxidation systems include flares and afterburners. A flare is basically an ignition chamber in which an ignitable gas is allowed to combust in a controlled air environment. The vent gases are ignited by use of a pilot burner. Smokeless flares have steam added, which converts any unburned heavy hydrocarbon to carbon dioxide and hydrogen (Liptak, 1974). Smokeless flares are usually not required for treating vent gases in waste disposal sites, since flares are used mainly for treatment of municipal landfill gases, which do not contain any hydrocarbons that generate smoke during combustion.

Afterburners are incinerators for gases and vapors in which fuel is burned to maintain a temperature of up to 1,600°F. Gases and vapors passing through the afterburner decompose in the presence of oxygen to carbon dioxide and water. Afterburners may incorporate the use of a catalyst, which facilitates oxidation at lower temperatures.

The above types of gas treatment systems will be discussed in the sections that follow.

6.5.2 Application

The specification of a particular type of gas treatment system depends on three site criteria: (1) type of contaminant; (2) its concentration in the overall gas stream; and (3) the total amount of gas being vented from the waste site.

The origin of the gas, and the components it contains, are essential criteria for the proper selection of a treatment. Gases from municipal landfills will be composed essentially of methane, hydrogen sulfide, hydrogen, and odorous volatiles such as butyric and proprionic acids. Methane is not considered a pollutant; however, it is vented to prevent lateral underground movement. Many times the accompanying gases and vapors are objectionable, and it is recommended that the gases be flared off (assuming they are not recovered as fuel gas). Flaring uses methane as fuel, and thus requires no auxiliary fuel input. Flaring can be applied to vent gases that are at, or above, the flammable concentration range. Gases with organic components that are present below flammability limits, or are not considered flammable, can be incinerated using fuel to generate the required temperatures. The device used for incineration of gases is usually referred to as an afterburner. Afterburners, if well designed and properly maintained, can achieve 98 percent destruction of pollutants (Liptak, 1974). Afterburners should only be used to treat gases and vapors that can be oxidized at temperatures of 1,600°F or less. In certain cases, where the contaminant is a relatively unstable chemical, a catalytic afterburner can be used with lower oxidation temperatures (1,000°F - 1,600°F). Generally, these thermal oxidation techniques should be restricted to those pollutants that will not produce objectionable oxidation products, such as chloride or fluoride generated from the oxidation of certain pesticides and freons.

Generally, flares and afterburners can be designed to handle a wide range of gas flow rates. Afterburners are more economical than the other techniques for gas treatment at high flow rates, or where high concentrations of combustibles are present in the gas stream.

Carbon adsorption systems are applicable to the treatment of vent gases containing large molecular weight organic components. Non-polar organic compounds are adsorbed best. Some high molecular weight organics are difficult to desorb from carbon, hence they exclude the use of regenerative systems. Carbon adsorption is also chosen when very toxic chemicals are present and the required removal efficiencies are greater than those obtainable with thermal oxidation. Carbon adsorption systems become expensive when they are used to control large gas volumes and high organic concentrations.

Carbon acts as a catalyst to oxidize hydrogen sulfide gas in municipal landfills. The efficiency of hydrogen sulfide adsorption can be increased by using activated carbon impregnated with metal oxides (Calgon, 1975).

6.5.3 Design and Construction Considerations

In the case of gas treatment systems, the actual design and construction is generally determined from specifications set by the equipment manufacturer. However, there are a number of important design and construction considerations with which the engineer working in remedial action technology should be familiar, to ensure that the equipment is properly suited for its application at the site, and for cost estimation.

With respect to treatment using air-activated carbon systems, an important decision must be made as to whether the systems will be non-regenerative or regenerative. If carbon is to be regenerated, a further decision must be made as to whether it is to be regenerated on- or off-site. Non-regenerative systems should be selected in cases where smaller gas volumes or low organic concentrations are involved, since the cost of disposal or incineration of spent carbon can be high. Also, if very toxic chemicals are present, such as dioxins, it may be best to use the non-regenerative system and dispose of (or incinerate) the spent carbon, rather than risk using inferior regenerated carbon with lower removal efficiencies (Liptak, 1974).

Regenerative systems may be more economical when treating larger gas streams. However, if the pollutant concentration is below 0.1 percent by volume, carbon regeneration is not economical and a non-regenerative system should be utilized (Rogoshewski et al., 1978). A regenerative system can incorporate the use of an off-site carbon regeneration unit, or regeneration may be part of the on-site installation. Usually, the off-site regeneration can be contracted to a vendor dealing with activated carbon (Calgon, 1980). A higher-quality regenerated carbon can be obtained by using off-site vendor-controlled regeneration rather than the less efficient on-site combined adsorption/regeneration process.

The maximum time period that a carbon bed may operate without a loss of efficiency can be calculated by using the following equation (Liptak, 1974).

$$t = \frac{SW}{MQC_v/RT}$$

where t = duration of adsorber, sec
S = fractional retentivity of adsorbent, mass adsorbate/mass adsorbent
W = mass of adsorbent in the bed, kg
M = molecular weight of adsorbate, kg/gmol
Q = volumetric flowrate of total gas, liters/sec
R = gas constant, 0.082 l-atm/gmol°K
T = temperature, °K
C_v = volume fraction of vapor in total gas

As shown above, there are a number of factors that affect the maximum operating time of a carbon adsorption bed. One of these factors, retentivity of adsorbent, is dependent on the type of substance being adsorbed. Thus, operating times are expected to be different for different pollutants, all other factors being constant. Table 6-6 lists several organic compounds with their retentivities before and after regeneration. Notice that retentivities are greatly reduced after regeneration, which confirms that regenerative adsorption systems are less efficient than non-regenerative beds. Also, in Table 6-6 the theoretical saturation rate is given for a number of compounds. In a carbon adsorption treatment system, solvent breakthrough occurs at a very low adsorbate/ adsorbent ratio. Thus, an equipment vendor usually uses a conservative adsorbate/ adsorbent ratio for design purposes. A typical design saturation fraction is about 15 percent (Calgon, 1980).

The above equation and the design saturation fraction can be used to determine the frequency of bed replacement, and also the amount of carbon that must be disposed or incinerated. Costs of operating a carbon adsorption treatment system can thus be determined.

The design of flare and afterburner systems is again primarily a function of the equipment vendor specifications. Flares for landfill gas will usually be the small ground flare type. Multiple flares connected in series can handle a large spectrum of gas flow rates. This is important to note, since gas generation rates will vary according to seasonal temperature, and proper generation of flares is limited to a narrow range of flow rates (Varec, Inc., 1980). Flares should be limited to low toxic gases, such as landfill methane.

Afterburners should be used when more toxic gases or vapors are encountered. It is important that the remedial action engineer consult suppliers about the suitability of gases not specifically recommended for their equipment. In some cases where corrosive oxidation products are formed, special

TABLE 6-6

RETENTIVITY BY ACTIVATED CARBON

Adsorbate	Saturation weight factor[1]	Approximate retentivity, weight fraction[2]	Retentivity after regeneration[3]
Benzene	0.45-0.55	0.25	0.06
Cabron tetrachloride	0.8-1.1	0.45	0.20
Gasoline	0.1-0.2	0.07	0.02
Methyl alcohol	0.5	--	0.01
Isopropyl alcohol	0.5	0.18	0.01
Ethyl acetate	0.58	0.20	0.05
Acetone	0.51	0.10	0.03
Acetic acid	0.70	0.30	0.03

[1] Weight of adsorbate per weight of dry carbon.
[2] Weight of adsorbate per wieght of carbon retained in dry air stream at 20°C.
[3] Regeneration with steam at 150°C for 1 hour.
(Source: Liptak, 1974)
See Copyright Notice, Page 496.

construction materials may be required, such as Monel* or Hastalloy* alloys. It is important to confirm that the contaminants in the gas stream can be oxidized at temperatures of 1600°F or less, and at retention times of 0.5 to 1 second, which represent operating ranges of afterburners. It is also important to consider the cost of fuel needed to maintain proper temperatures before this treatment plan is selected.

6.5.4 Advantages and Disadvantages

Tables 6-7, 6-8, and 6-9 give a summary of advantages and disadvantages of carbon adsorption, flares, and afterburners, respectively.

6.5.5 Costs

The capital and operating costs for carbon adsorption, flares, and afterburners are presented in the following tables and graphs. Capital and operating costs for non-regenerative carbon adsorption treatment systems were

---

*These are trademark names.

TABLE 6-7

ADVANTAGES AND DISADVANTAGES OF CARBON ADSORPTION GAS TREATMENT SYSTEMS

Advantages	Disadvantages
● Removes most organic compounds from gas streams	● Will not remove polar organics efficiently
● Very high removal efficiencies	● Not compatible with methylethyl ketone
● Able to oxidize hydrogen sulfide	● Requires constant monitoring system
● Applicable to large gas streams with low organic concentration	● Not applicable to municipal landfill gases since methane is not adsorbed
	● Does not remove light organics effectively
	● Requires either regeneration or bed replacement
	● Requires disposal of spent carbon or desorbed material

TABLE 6-8

ADVANTAGES AND DISADVANTAGES OF FLARE SYSTEMS

Advantages	Disadvantages
● Applicable to combustible gases at, or above, the flammability threshold	● Not applicable to dilute gas streams below flammability threshold
● Low capital and operating cost	● Not as efficient as incineration or carbon adsorption
● Most applicable to flaring of low toxicity gases or vapors, such as those from municipal landfills	● Narrow range of flow rates results in requirement for more than one flare

TABLE 6-9

ADVANTAGES AND DISADVANTAGES OF AFTERBURNERS

Advantages	Disadvantages
● High removal efficiency	● Large fuel cost
● Able to destroy almost all organics	● Expensive for small gas streams
	● May not be able to destroy organics that will not oxidize in less than 1 second at temperatures below 1600°F
	● May form corrosive oxidation products and may, therefore, require expensive construction materials
	● May generate smoke if heavy unsaturated hydrocarbons are present

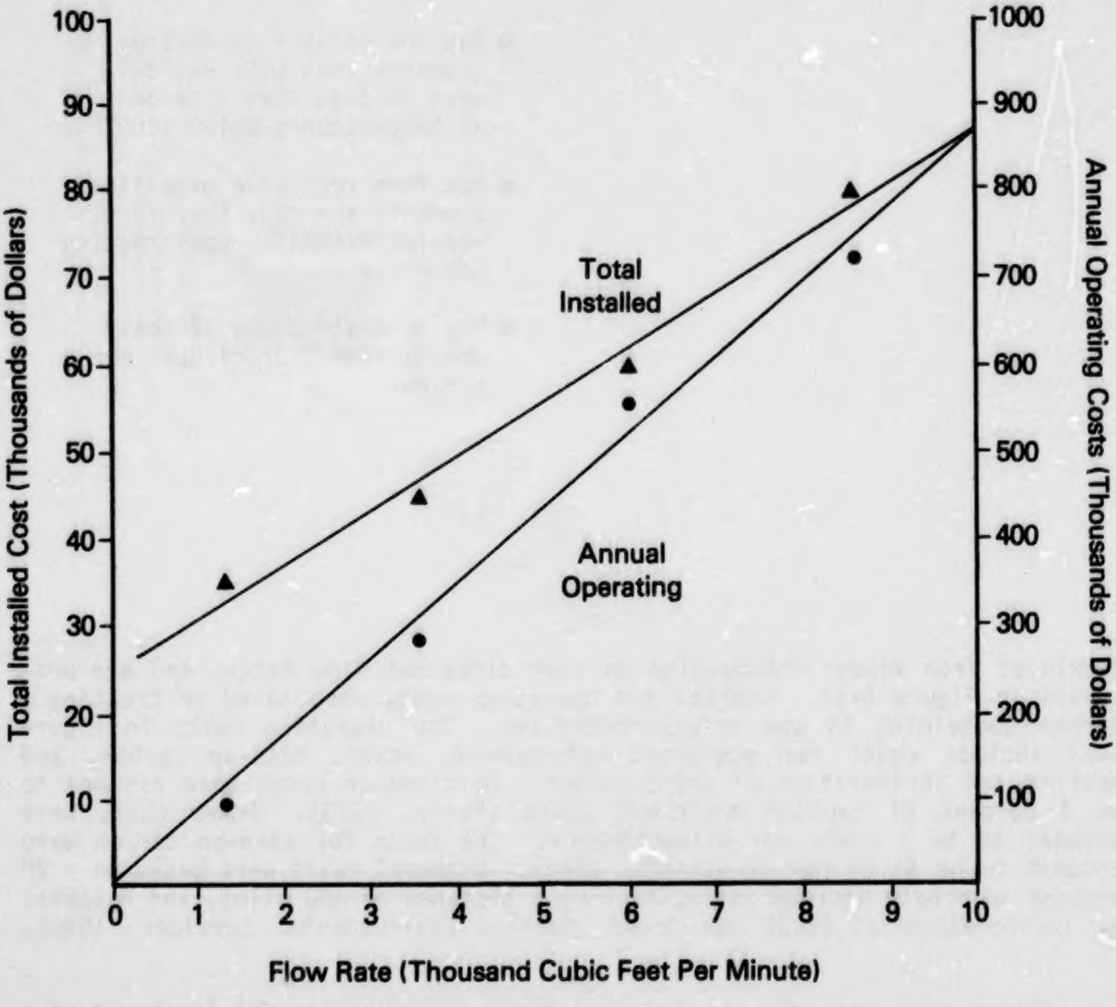
developed from vendor information on four different flow rates, and are presented in Figure 6-13. Capital and operating costs were based on treating a stream containing 50 ppm trichloroethylene. The operating costs in Figure 6-13 include costs for equipment maintenance, power, make-up carbon, and hauling and incineration of spent-carbon. Maintenance costs were assumed to be 4 percent of capital equipment costs (Perry, 1973). Power costs were assumed to be 7 cents per kilowatt-hour. The costs for make-up carbon were assumed to be $1.20 per lb (Calgon, 1980). Disposal costs were based on a 20 percent adsorbate pick-up ratio, hauling a distance of 400 miles, and disposal by incineration at $0.25 per pound (Rollins Environmental Services, 1980).

Figure 6-13 can only be used as a rough approximation for treatment of a vent gas, since the information was developed for a stream having a specific pollutant at a specific concentration. It will be necessary to work with vendors to determine the design and costs of a non-regenerative carbon unit for a particular application.

FIGURE 6-13

CAPITAL AND OPERATING COSTS FOR NON-REGENERATIVE CARBON ADSORPTION SYSTEMS TREATING VENT GAS CONTAINING 50 ppm TRICHLORETHYLENE

(Source: Calgon, 1980)



Capital costs for regenerative carbon systems are higher. The installed cost for a 1,700 ft^3/min unit was quoted at approximately $140,000 (Baron-Blakeslee, 1980). This high cost is due to the extra equipment needed to regenerate the carbon, plus special construction materials needed to prevent corrosion at regeneration temperatures. Operating costs will be much less than non-regenerative systems, since the high cost of carbon make-up and disposal is not involved. Annual operating costs have been estimated to be roughly $12,000 per year for the 1,700 ft^3/min unit (Baron-Blakeslee, 1980).

Costs for flare systems have also been developed for a number of flow rates. Total costs are given in Figure 6-14, and include material costs, plus 25 percent for installation, contractor overhead, and profit. Operating costs are assumed to be negligible.

Installed costs for afterburning vary widely, and are presently estimated to range from $5 to $20 per standard cubic foot per minute (Liptak, 1974). The high end of the range is more applicable to the treatment of vent gases from waste disposal sites, since the contaminants in the vent gas are likely to be corrosive, thus requiring special construction materials. Annual operating costs for an afterburner with heat recovery, have been estimated to range from $10 to $40 per cubic foot per minute (Combustion Engineering, 1980). Catalytic incinerators are usually associated with lower costs for operation as they are able to use the vent gas as a fuel source.

6.6 GAS RECOVERY

As mentioned earlier, it may be possible to recover methane gas and possibly other gases from landfills. Gas recovery is not a remedial action, per se, but it may be included as part of a gas migration control plan, and may partially offset the initial capital investment for gas control in the long term. It is most applicable to landfills with a minimum depth of 30 to 40 feet. Otherwise, withdrawal rates must be limited to prevent air infiltration, and the economics of recovery may suffer (James and Rhyne, 1979).

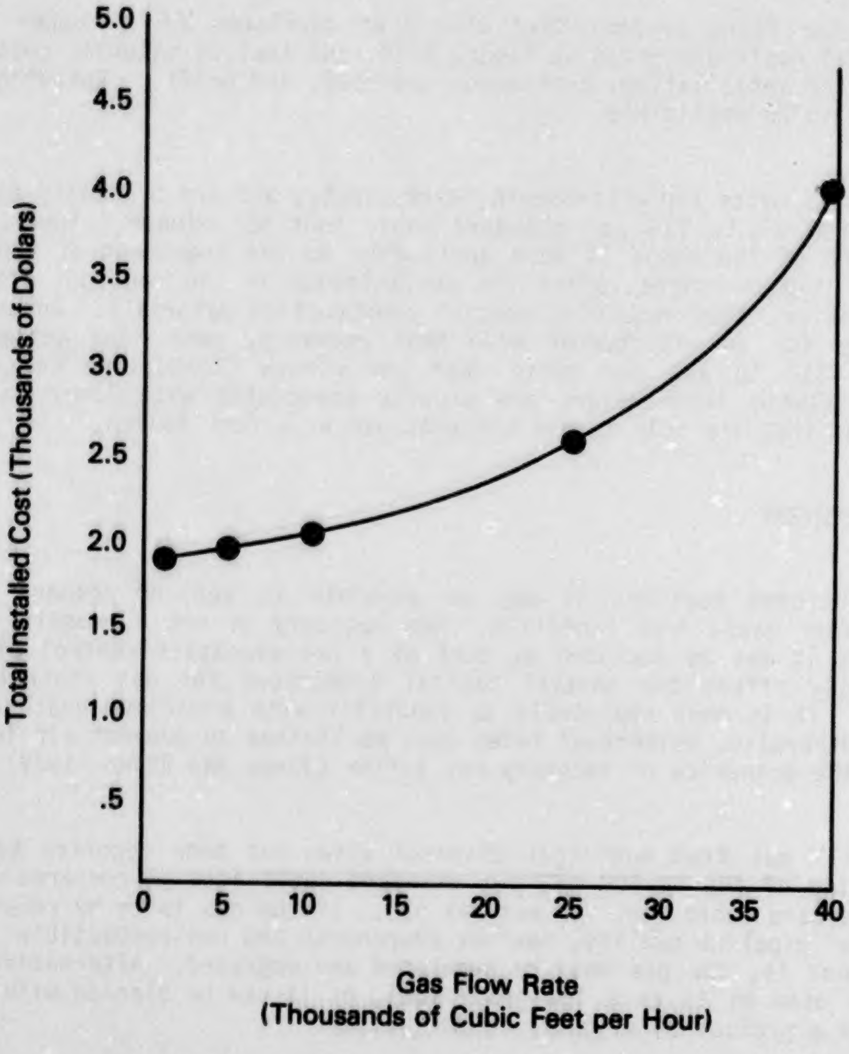
Landfill gas from municipal disposal sites has been reported to have a heating value of 450 to 500 BTU per standard cubic foot as compared to 1,000 BTU per standard cubic foot for natural gas. If the gas is to be recovered to the point of pipeline quality, noxious components and non-combustibles must be removed, that is, the gas must be sweetened and upgraded. Alternatively, the gas can be used as is as a low grade fuel, or it can be blended with natural gas to form a product of slightly lower quality.

The Palos Verde Gas Recovery Project, sponsored by Reserve Synthetic Fuels, used a molecular sieve treatment facility to sweeten and upgrade and landfill gas from 500 to 1000 BTU per standard cubic foot (James and Rhyne, 1979). It was reported that severe corrosion occurred in the recovery system

FIGURE 6-14

TOTAL INSTALLED COST FOR SMALL GROUND FLARES

(Source: Varec, Inc., 1980)



due to the presence of chlorinated hydrocarbons. This was corrected by using corrosion-resistant nickel alloys and pretreating with a material to selectively adsorb the corrosives.

Various upgrading schemes are available, from simple dehydration to a combined dehydration, carbon dioxide, and nitrogen removal system. The more contaminants removed, the greater the heating value of the recovered gas and the greater the production cost. Another gas recovery project at the municipal landfill at Mountain View, California, plans to upgrade gas from 450 BTU/scf to 700 BTU/scf and blend this with pipeline natural gas (James and Rhyne, 1979). In the process, dehydration, carbon dioxide removal, and molecular sieves will be used. Costs for a 1-million cubic foot per day recovery plant have been estimated, based on an overall efficiency of 70 percent, 12 percent cost of capital, a 10-year life, and a salvage value of 30 percent (James and Rhyne, 1979). These costs are presented in Table 6-10, which also include costs for the gas venting and collection system.

TABLE 6-10

COSTS FOR LANDFILL GAS RECOVERY AT MOUNTAIN VIEW, CALIFORNIA[1]

	Equipment cost	Installed Cost
Molecular sieves	$245,000	$368,000
Compression	200,000	350,000
Wells and gathering system	- - - -	70,000
Total installed cost		$788,000

Yearly costs	$/Year
Maintenance	25,000
Manpower	30,000
Fixed charges	195,000
Feedstock costs	22,320
Total	272,320
Energy output, MMBTU/yr	97,650
Energy costs, $/MMBTU	$2.79

---

[1] Based on a flow of 1 million ft^3/day
(Source: James and Rhyne, 1979)

REFERENCES

Aladdin Heating Corporation. 1980. Industrial Tans Series 2000. San Leandro, CA. Bulletin 462-C

ARMCO, Inc., Baltimore, MD. April 1980. Personal communication with P. Le.

Baron-Blakeslee Corp., Bayshore, NY. April 1980. Personal communication with P. Rogoshewski.

B.F. Goodrich, Inc. 1980. Information Bulletin EL-1.5-775. B.F. Goodrich Environmental Products.

Bowerman, F. Engineering Science, Inc. Arcadia, CA. March 1980. Personal communication with P. Rogoshewski.

Britton Company, Norfolk, VA. April 1980. Personal communication with P. Rogoshewski.

Bureau of Reclamation. 1977. Groundwater manual. U.S. Department of Interior, Denver, CO.

Calgon Corp. 1975. Air purification with granular activated carbon. Pittsburgh, PA.

Calgon Corp., Pittsburgh, PA. March 1980. Personal communication with P. Rogoshewski.

Camwell Corp., Baltimore, MD. April 1980. Personal communication with P. Rogoshewski.

Carlson, J. 1977. Recovery of landfill gas at Mountain View: Engineering site study. U.S. Environmental Protection Agency, Washington, D.C. EPA/530/SW-587d.

Combustion Engineering, Wellsville, NY. April 1980. Personal communication with P. Rogoshewski.

Constable, T., G. Farquhar, and B.N. Clement. 1979. Gas migration and modeling. In: Proceedings of the 5th annual research symposium on municipal solid waste: land disposal. U.S. Environmental Protection Agency, Washington, D.C. EPA-600-9-79-023a.

DuPont de Nemours & Co., Inc. 1980. Flexible membranes for pond and reservoir liners and covers. Wilmington, DE.

Farmer, W., M. Yang, and J. Lefey. 1978. Land disposal of hexachlorobenzene wastes: controlling vapor movement in soils. Land disposal of hazardous wastes. Municipal Environmental Research Laboratory. Cincinnati, OH. EPA-600/9-78-016.

Godfrey, R. (ed.) 1979. Building construction cost data, 1980. Kingston, MA: Robert Snow Means Company, Inc.

James, C., and C. Rhyne. 1979. Methane production, recovery, and utilization from landfills. Recovery, Processing, and Utilization of Gas from Sanitary Landfills. U.S. Environmental Protection Agency, Cincinnati, OH. EPA-600-2-79-001.

JRB Associates, and EMCON Associates. 1980. Assessment of alternatives for upgrading Navy solid waste disposal sites: draft final report. McLean, VA.

Leazer Pumps and Wells, Remington, VA. April 1980. Personal communication with K. Wagner.

Liptak, B.G. (ed.) 1974. Environmental engineers' handbook. Vol. 1: Air pollution; vol. 3: Land Pollution. Radnor, PA: Chilton Book Co.

Lutton, R., G. Regan, and L. Jones. 1979. Design and construction of covers for solid waste landfills. U.S. Environmental Protection Agency, Cincinnati, OH, EPA-600/2-79-165.

McCaffray Company, Batimore, MD. April 1980. Personal communication with P. Rogoshewski.

McMahon, L., and P. Pereira (eds.). 1979. 1980 Dodge guide to public works and heavy construction costs. New York: McGraw-Hill Information Systems.

Moore, C.A. 1976. Theoretical approach to gas movement through soils. Gas and leachate from landfills, formation, collection. U.S. Environmental Protection Agency, Cincinnati, OH. EPA-600-19-76-004.

Moore, C.O., and I. Rai. 1977. Design criteria for gas migration control devices. Mangement of gas and leachate in landfills. U.S. Environmental Protection Agency, Cincinnati, OH. EPA-600/9-77-026.

Perry, R., and C. Chilton. 1973. Chemical engineer's handbook, 5th ed. New York: McGraw-Hill, Inc.

Plastic Piping Systems, Inc., Columbia, MD. April 1980. Personal communication with P. Rogoshewski.

Rogoshewski, P., P. Koester, C. Koralek, R. Wetzel, and K. Shields. 1978. Standards of practice manual for the solvent refined coal liquefaction process. U.S. Environmental Protection Agency, Washington, D.C. EPA-600-7-78-091.

Rollins Environmental Services, Bridgeport, NJ. April 1980. Personal communication with P. Rogoshewski.

Shen, T., and T. Tofflemire. 1980. Air pollution aspects of land disposal of toxic waste. Hazardous materials risk assessment, disposal and management. Journal of Environmental Engineering Division, ASCE 106(EEI): 211-226.

SMACNA. 1967. Manual for the balancing and adjustment of air distribution systems. Sheet Metal and Air Conditioning Contractors National Association, Inc., Vienna, VA.

Staples, G., JRB Associates, McLean, VA. April 1980. Personal communication with P. Le.

Stone, R. 1978a. Preventing the underground movement of methane from sanitary landfills. Civil Engineering. January.

Stone, R. 1978b. Reclamation of landfill methane and control of off-site migration hazards. Solid wastes management/refuse removal journal. July.

Thibodeaux, L. 1979. Estimating the air emissions of chemicals from hazardous waste landfills. American Institute of Chemical Engineers annual meeting, San Francisco, CA.

Trane Company. 1965. Trane air conditioning manual. LaCrosse, WI.

Universal Linings, Inc., Philadelphia, PA. 1980. Personal communication between D. Small and P. Rogoshewski.

U.S. Department of Interior. 1977. Groundwater manual. Bureau of Reclamation. Denver, CO.

Varec, Inc., Cherry Hill, NJ. April 1980. Personal communication with P. Rogoshewski.

7.0 DIRECT TREATMENT METHODS

This chapter discusses the direct treatment of hazardous wastes for purposes of alleviating environmental or health risks associated with these wastes. The single most important advantage of direct treatment methods as compared to other methods discussed in this document is that direct treatment affords removal of pollution at the source, while other methods simply contain. In most cases, these techniques can be considered long-term permanent solutions. Direct treatment of hazardous wastes involves one or more of the following approaches:

- Physical removal of the wastes to a better engineered or environmentally less sensitive area (excavation, hydraulic dredging, and land disposal);
- Physical removal followed by waste stabilization (solidification and encapsulation);
- Waste destruction (incineration, wet air oxidation, molten salt, macrowave plasma detoxification. and microbial degradation);
- Chemical waste treatment within the site (neutralization and solution mining).

Many of these direct treatment methods are not fully developed and the applications and process reliability are not well demonstrated. Use of these techniques for waste treatment will require considerable pilot plant work. Others, such as excavation and land disposal, are widely used, although Resource Conservation and Recovery Act (RCRA) requirements will require significantly greater degrees of monitoring and controls for land disposal techniques. In addition, the combined costs of excavation (or dredging) with subsequent treatment can be exorbitant if a large volume of wastes is involved.

This chapter addresses the current state of technology and applicability of each of the direct treatment methods.

7.1 EXCAVATION

7.1.1 General Description and Applications

Excavation is a common technique used in earth-moving projects. It is widely used to move solid and thickened sludge materials; however, it is not well suited for removal of material with a low solids content. Where off-site treatment methods are to be used for landfilled wastes, excavation and transportation of the waste material will be required.

7.1.2 Design and Construction Considerations

Important factors that should be considered before excavation of a refuse site can begin are listed below.

1. The density of solid waste in a landfill. This is dependent on the composition of the waste and the degree of compaction achieved. Average densities of landfilled wastes are reported to be from 800 to 1,000 lb/yd^3 with moderate compaction (Brunner and Keller, 1972).

2. The settlement of the fill. As a result of decomposition of the waste and the addition of new waste material, settling of fine particles into voids between solid matter can occur.

3. The bearing capacity of the fill. The bearing capacity is defined as the ability to support foundations (and heavy equipment). Although the bearing capacity of the fill can vary from one refuse site to another, average values ranging from 500 to 800 lb/ft^2 have been reported (EPA, 1978).

4. Decomposition rate of the waste. Most of the materials present in a refuse site will decompose. Decomposition of organic waste under anaerobic conditions predominantly occurs at the base of the site, and can generate highly corrosive organic acids and toxic gases such as methane or hydrogen sulfide.

5. Packaging of the waste. Packaging of waste in barrels and tanks may present additional removal problems.

Excavation can be achieved by mechanical means. Typical excavation equipment includes draglines and backhoes. This equipment is discussed below.

7.1.2.1 The Dragline

A dragline excavator is a crane unit with a drag bucket connected by cable to the boom. The bucket is filled by scraping it along the top layer of soil toward the machine by a drag cable. The dragline can operate below and beyond the end of the boom. Figure 7-1 shows a dragline unit. The various working dimensions of draglines for various bucket sizes are shown in Table 7-1.

Maximum digging depth of a dragline is approximately equal to half the length of the boom, while digging reach is slightly greater than the length of the boom (EPA, 1978). Drag buckets come in light, medium, and heavy weight. The use of a specific weight of bucket depends on the type of material to be excavated.

The maximum practical digging depths for various lengths of boom are shown in Figure 7-2. As shown in this figure, the recommended slope for dragline is 45°. With a 70 foot boom, the maximum digging depth is approximately 60 feet. Draglines are very suitable for excavating large land areas with loosely compacted soil. Excavation with draglines of landfill sites containing explosive materials or very toxic chemicals is unsafe.

The theoretical hourly production rate in cubic yards for operating in different types of soil with various bucket sizes is shown in Table 7-2. The figure presented in this table is estimated on the basis of 83 percent job efficiency, 100 percent operator efficiency, 90 percent swing of boom, and 50 working minutes per hour (Godfrey, 1976). The actual production is estimated at 50 percent of the theoretical values. As shown in this table, the optimum digging depth of any size bucket is less than 14 feet.

Advantages, disadvantages and costs for this excavation method are given in Section 7.1.3. and 7.1.4.

7.1.2.2 The Backhoe

The backhoe unit is a boom or dipper stick with a hoe dipper attached to the outer end. The unit may be mounted on either crane-type or tractor equipment. Figure 7-3 shows a typical design of a backhoe.

The digging dimensions of a backhoe are shown in Figure 7-4. The maximum reach and depth for various sized hoes is shown in Table 7-3. As shown in this figure, the largest backhoe will dig to a maximum depth of about 30 feet. Deeper digging depth can be achieved by attaching long arms to one-piece booms, or by adjusting the boom angle on two piece booms (EPA, 1978).

FIGURE 7-1

A DRAGLINE

(Source: EPA, 1978)

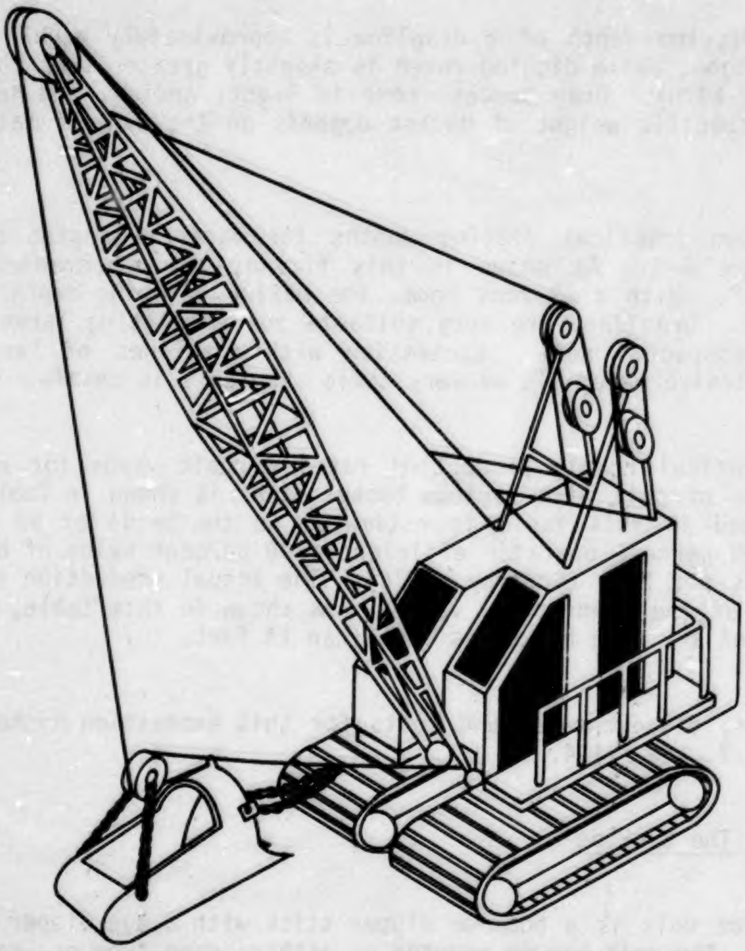


TABLE 7-1

TYPICAL DRAGLINE EXCAVATOR DIMENSIONS

Item	Bucket size in cubic yards (CY) 3/4	1	1-1/4	1-3/4	2
Dumping radius, ft	30	35	36	45	53
Dumping height, ft	17	17	17	25	28
Maximum digging depth, ft	12	16	19	24	30
Digging reach, ft	40	45	46	57	68
Boom Length, ft	35	40	40	50	60
Bucket length, ft	11.5	14.67	11.83	13.08	14

Note that these values apply to operation of the excavator with its boom at a 40° angle to the horizon.

Source: EPA, 1978

Another hydraulic backhoe called the Gradall can be used to excavate, backfill, and grade. It has an extensional boom which can be extended up to 100 feet or retracted for close work.

The theoretical hourly production rate for a backhoe for a 15 foot-deep cut in different types of soil with various bucket sizes is shown in Table 7-4. The bases for these estimates are the same as those presented for the dragline production.

To achieve deeper digger depth (i.e., deeper than 30 feet), clamshell equipment must be used. A clamshell bucket is attached to a crane by cables. A clamshell excavator can reach a digging depth of 50 feet or more. Figure 7-5 illustrates a clamshell bucket.

Regardless of the type of excavation equipment used, safety considerations must be taken into account when the excavated sites contain toxic substances or explosive materials. The safety measures are discussed in Appendix A.

FIGURE 7-2

TYPICAL WORKING RANGES FOR CRANES AND DRAGLINES

(Source: Stubbs, 1959)
See Copyright Notice, Page 496

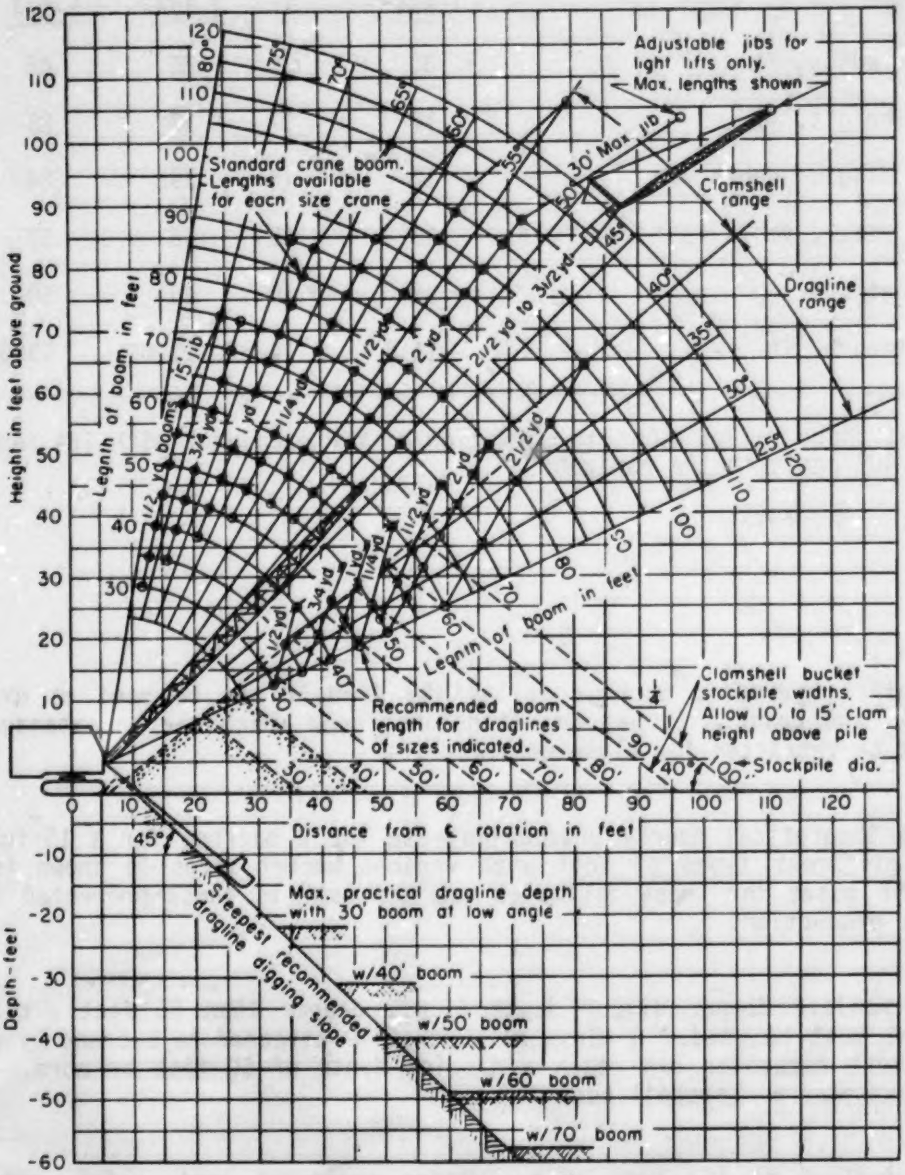


TABLE 7-2

THEORETICAL HOURLY PRODUCTION RATE OF A DRAGLINE EXCAVATOR

	Bucket size in cubic yards (CY)						
Type of soil	1	1-1/2	2	2-1/2	3	3-1/2	4
Moist loam, sandy clay	130 (6.6)	180 (7.4)	220 (8.0)	250 (8.5)	290 (9.0)	325 (9.5)	385 (10.0)
Sand and gravel	130 (6.6)	175 (7.4)	210 (8.0)	245 (8.5)	280 (9.0)	315 (9.5)	375 (10.0)
Common earth	110 (8.0)	160 (9.0)	190 (9.9)	220 (10.5)	250 (11.0)	280 (11.5)	375 (10.0)
Clay, hard dense	90 (9.3)	130 (10.7)	160 (11.8)	190 (12.3)	225 (12.8)	250 (13.3)	280 (12.0)

Note: Numbers in parentheses represent the maximum digging depth.

Source: EPA, 1976

FIGURE 7-3

TYPICAL BACKHOE

(Source: EPA, 1978)

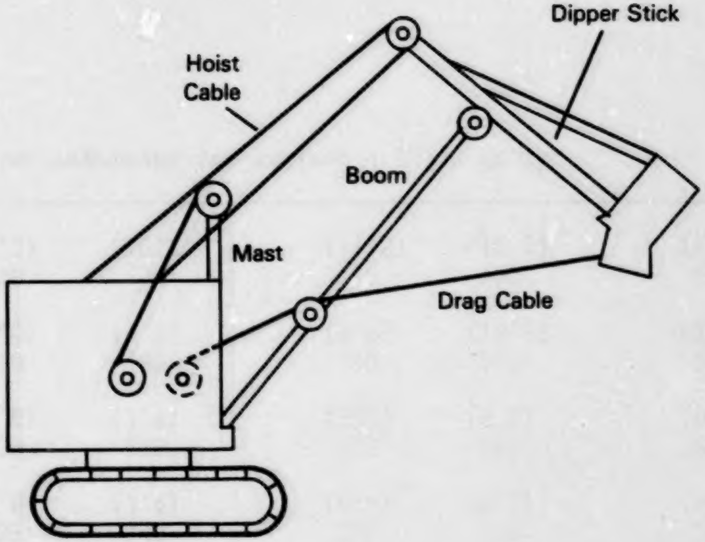

Dipper Stick
Hoist Cable
Boom
Mast
Drag Cable

FIGURE 7-4

HOE DIGGING RANGES

(Source: Stubbs, 1959)
See Copyright Notice, Page 496

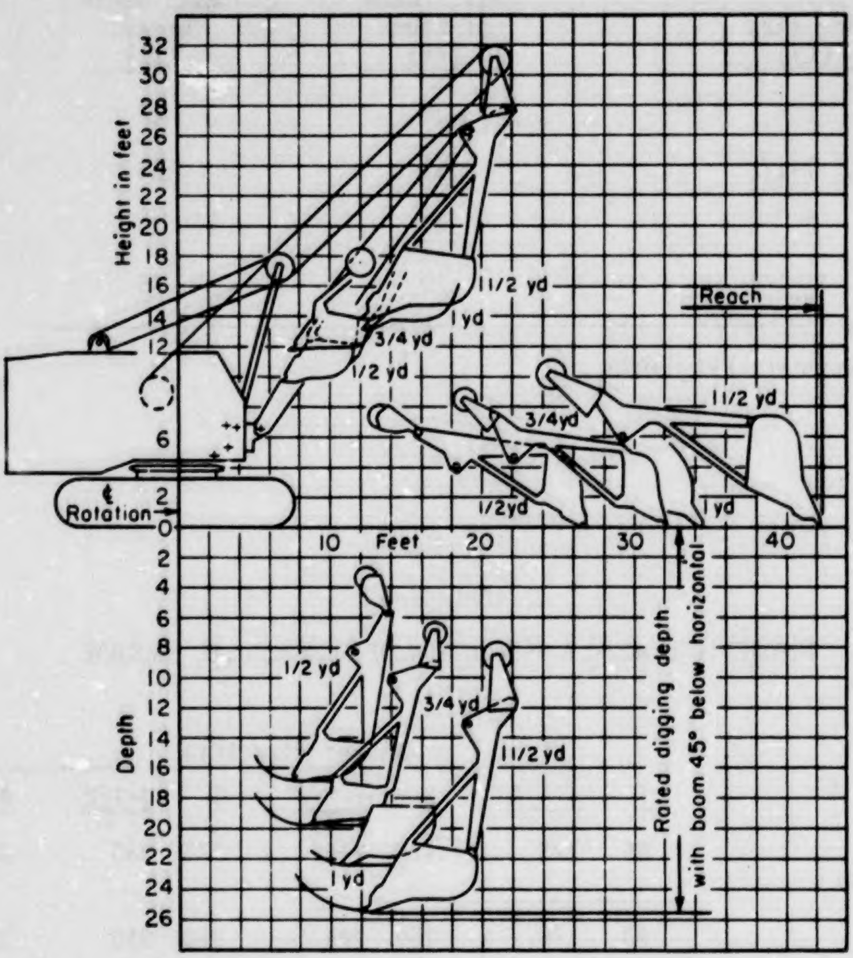

Height in feet
32
30
28
26
24
22
20
18
16
14
12
6
2
0
℄ Rotation
10
Feet
20
30
40
2
4
6
8
10
12
14
16
18
20
22
24
26
Depth
11/2 yd
1 yd
3/4 yd
1/2 yd
Reach
11/2 yd
3/4yd
1/2yd
1 yd
1/2 yd
3/4 yd
11/2 yd
1 yd
Rated digging depth
with boom 45° below horizontal

TABLE 7-3

MAXIMUM REACH AND DEPTH FOR VARIOUS SIZED HOES

(MAXIMUM DIGGING ANGLE OF 45°)

Hoe size (CY)	Max. reach of boom (ft)	Max. depth excav. (ft)
1	35	22
1-1/2	42	25
2	49	30
3-1/2	70	45

Source: EPA, 1978

TABLE 7-4

THEORETICAL HOURLY PRODUCTION OF A HYDRAULIC BACKHOE

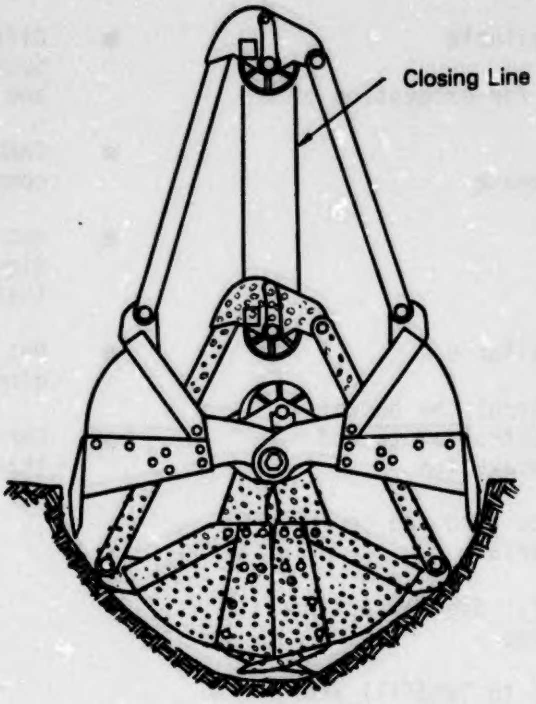
	Bucket size (CY)						
	1	1-1/2	2	2-1/2	3	3-1/2	4
Moist loam, sandy clay	85	125	175	220	275	330	380
Sand and gravel	80	120	160	205	260	310	365
Common earth	70	105	150	190	240	280	330
Clay, hard dense	65	100	130	170	210	255	300

Source: EPA, 1978

FIGURE 7-5

THE CLAMSHELL BUCKET

(Source: Carson, 1961)
See Copyright Notice, Page 497



7.1.3 Advantages and Disadvantages

Advantages and disadvantages of the excavation technique using dragline and backhoe are shown in Table 7-5.

7.1.4 Costs

Unit costs for excavation techniques are shown in Table 7-6.

TABLE 7-5

ADVANTAGES AND DISADVANTAGES OF DIFFERENT TYPES OF EXCAVATORS

Advantages

Dragline

- Readily available
- Applicable for excavation of large area
- Easy to operate

Backhoe

- Readily available
- Easy to control the bucket and thus control width and depth of excavation
- Can excavate hard and compacted material
- More powerful digging action than dragline
- Can be used to landfill and compact

Disadvantages

- Difficult to spot bucket for scraping and dumping
- Cannot backfill or compact
- Not applicable for digging depth more than 30 ft
- Not applicable for digging depth over 30 ft
- Cannot reach further than 100 ft

The excavation cost of a refuse site can be estimated as follows, assuming that the dimensions of the site are 30 feet deep, 1,000 feet long, and 750 feet wide, then the total volume of waste excavated would be:

$$(30\ \text{ft})\ (1000\ \text{ft})(750\ \text{ft}) = 2.25 \times 10^7\ \text{ft}^3 \text{ or } 8.3 \times 10^5\ \text{yd}^3$$

- Using a dragline equipped with a 1.5 yd^3 bucket, it would cost $\$1.39/yd^3$ (Table 7-6). Thus the excavation cost amounts to:

$$(8.3 \times 10^5\ \text{yd}^3)(\$1.39/\text{yd}^3) = \$1{,}160{,}000$$

- Using a backhoe equipped with a 1.5 yd^3 bucket, the total excavation cost would be:

  $(8.3 \times 10^5\ \text{yd}^3)(\$1.51/\text{yd}^3) = \$1,250,000$

- Using a clamshell unit with a 1 yd bucket, it would cost:

  $(8.3 \times 10^5\ \text{yd})\ (\$2.32/\text{yd}^3) = \$1,930,000$

7.2 HYDRAULIC DREDGING

7.2.1 Description and Applications

For unlined surface impoundments containing hazardous wastes in liquid, slurry, or semi-solid (sludge) form, it may be necessary to remove the wastes by dredging. Several types of dredges are commonly used, including hydraulic, pneumatic, and mechanical dredges (the latter of which include clamshells, backhoes, and buckets, previously described in Section 7.1). This section addresses hydraulic dredges.

The dredged wastes can be pumped to special treatment facilities or transported to acceptable land disposal sites located nearby. The dredged impoundment can be bottom-lined and, if necessary, reconstructed to make it suitable for accepting industrial hazardous wastes. If the impoundment is located in a totally unacceptable area (e.g., wetlands or floodplain), it may be filled in and never reused.

Surface impoundments for which hydraulic dredging may be prescribed include industrial storage, treatment, and disposal ponds -- holding ponds, settling ponds, aeration lagoons, sludge or slurry pits, dewatering basins, etc. These surface impoundments may be natural depressions, artificial excavations, or diked containment areas. If the impoundments are outlined or if the liner is ruptured or torn, and the contained wastes are hazardous in nature, the potential of groundwater contamination by hazardous leachate may exist. Available techniques for hydraulic dredging of surface impoundments include centrifugal pumping systems and portable hydraulic pipeline dredges. Hydraulic dredging serves the same basic function as mechanical excavation: removal of hazardous waste materials from improperly constructed and improperly sited disposal sites for offsite treatment or disposal.

Centrifugal pumping systems utilize specially designed centrifugal pumps that chop and cut heavy, viscous materials as pump suction occurs. The special chopper-impeller devices within these pumps allow high-volume handling of heavy sludges and other solids mixtures without the use of separate augers or cutters (Vaughan, 1980). These submersible pumps are installed on floating, winch-driven platforms that can quickly and economically dredge

TABLE 7-6

UNIT COST FOR EXCAVATION[1]

	Assumption	Cost
Excavation using dragline	3/4 yd^3 bucket, 90° swing, rating 35/hr	$\$2.06/\text{yd}^3$
	1.5 yd^3 bucket, 90° swing, rating 65 yd^3/yr	$\$1.39/\text{yd}^3$
Excavation using backhoe	hydraulic, crawler mounted	
	- 1 yd^3 bucket, rating 45 yd^3/hr	$\$1.78/\text{yd}^3$
	- 1.5 yd^3 bucket, rating 60 yd^3/hr	$\$1.51/\text{yd}^3$
	- 2 yd^3 bucket, rating 75 yd^3/hr	$\$1.59/\text{yd}^3$
	- 3.5 yd^3 bucket, rating 150 yd^3/hr	$\$1.16/\text{yd}^3$
	wheel mounted	
	- 0.5 yd^3 bucket, rating 20 yd^3/hr	$\$3.24/\text{yd}^3$
	- 0.75 yd^3 bucket, rating 30 yd^3/hr	$\$2.48/\text{yd}^3$
Excavating using clamshell	0.5 yd^3 bucket, rating 20 yd^3/hr	$\$3.47/\text{yd}^3$
	1 yd^3 bucket, rating 35 yd^3/hr	$\$2.32/\text{yd}^3$

---

[1] Godfrey, 1979.

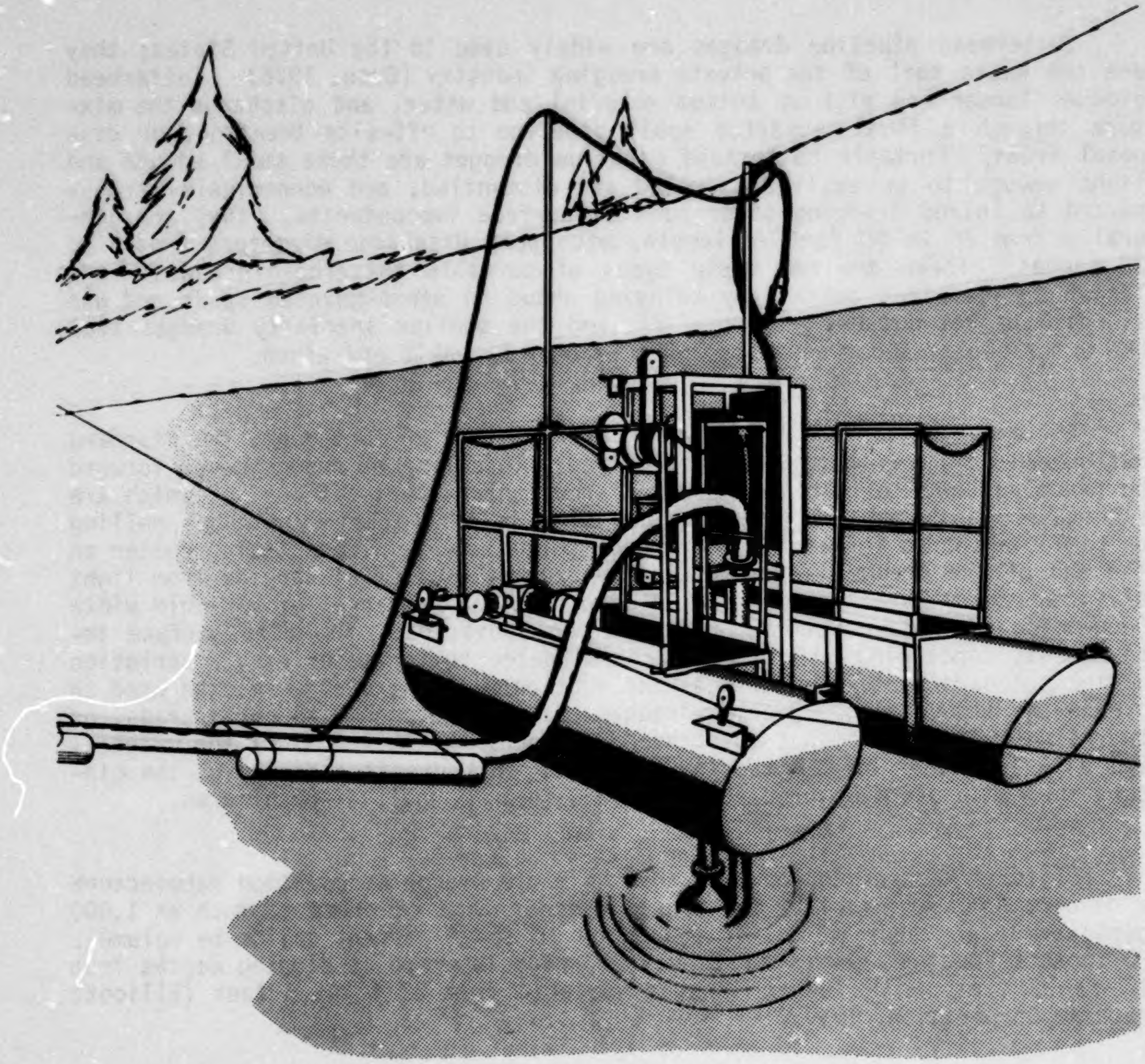
small pits, ponds, or lagoons. The Vaughan Company's "Lagoon Pumper" (Figure 7-6) is 8 feet wide, 14 feet long, approximately 7 feet high, and weighs about 3 tons; its 100-horsepower motor can pump up to 1,200 gpm of 15 to 20 percent solids from depths up to 15 feet (Vaughan, 1980).

National Car Rental systems manufactures a similar unit, the Mud Cat model SP-810, that utilizes a submerged pump mounted directly behind a horizontal auger to handle highly viscous chemical sludges or thick, muddy sediments. The SP-810's centrifugal pump is hydraulically driven, has a variable speed capability, and can pump from a maximum depth of 10 feet at reates up to 1,000 gpm. As with the Vaughan unit, the pump can be physically buried into homogeneous settled sludges for high solids/low dilution pumping. The Mud Cat unit also has a detachable mud shield for greater suction efficiency and turbidity control. It is equipped with a depth gage to monitor cutting depth, and is driven along a cable by a reversible winch (National Car Rental System, Inc., 1980).

FIGURE 7-6

PORTABLE CENTRIFUGAL PUMP SYSTEM FOR LAGOON DREDGING

(Source: Vaughan, 1980)



These centrifugal pumping systems are relatively small, portable units that may be ideal for small impoundment dredging where depths are less than 15 feet, where highly viscous materials such as consolidated chemical sludges are present, and where direct truck loading of dredged material without dewatering is desirable. For larger impoundments that require greater operational depths, higher volume removal, higher pumping rates, and greater pumping distances, larger (but still portable) dredge vessels are required.

Cutterhead pipeline dredges are widely used in the United States; they are the basic tool of the private dredging industry (Gren, 1976). Cutterhead dredges loosen and pick up bottom material and water, and discharge the mixture through a float-supported spoil pipeline to off-site treatment or disposal areas. Portable cutterhead pipeline dredges are those small enough and light enough to be easily assembled and dismantled, and economically transported to inland dredging sites such as surface impoundments. They are generally from 25 to 60 feet in length, with pump discharge diameters from 6 to 20 inches. There are two basic types of portable cutterhead dredges: the larger dredges that operate by swinging about on stern-mounted spuds and use standard basket cutters (Figure 7-7); and the smaller specialty dredges that use a horizontal auger assembly and move only by cable and winch.

For dredging surface impoundments greater than 20 feet deep, the standard cutterhead dredge (Figure 7-8) is required. This type of dredge moves forward by pivoting about on two rear-mounted spuds (heavy vertical posts), which are alternately anchored and raised. The swing is controlled by winches pulling on cables anchored forward of the dredge (Figure 7-9). The rotating cutter on the end of the dredge ladder physically excavates material ranging from light silts to consolidated sediments or sludge, cutting a channel of variable width (depending on ladder length) as the dredge advances. For deep surface impoundments containing only soft, unconsolidated bottom materials, a variation of the standard cutterhead dredge--the suction pipeline dredge--can be used to dredge the impoundment. Suction dredges are not equipped with cutterheads, or they simply operate without cutterhead rotation; they merely suck the material and dilution water off the bottom and, like most dredges, discharge the mixture through a stern-mounted pipeline leading to a spoil disposal area.

Ellicott Machine Corporation and the Dixie Dredge Corporation manufacture a diverse line of portable cutterhead dredges that can pump as much as 1,000 cubic yards per hour of solids (based on 10 to 20 percent solids by volume). Ellicott's "Dragon" series of portable dredges operate at digging depths from 17 to 33 feet, with ladder lengths variable from 23 to 42.5 feet (Ellicott Machine Corporation, 1969).

For hydraulic dredging of surface impoundments less than 20 feet deep, where access for the larger portable cutterhead dredges may be difficult, and where turbidity (material re-suspension) is a problem, small cutterhead dredges with horizontal auger cutters may be effective. National Car Rental's Mud Cat MC-915 (Figure 7-10) can remove sediment in a 9-foot-wide swath, 18

FIGURE 7-7A

STANDARD CUTTER ASSEMBLY: SPIRAL BASKET CUTTER

(Source: Huston, 1976)

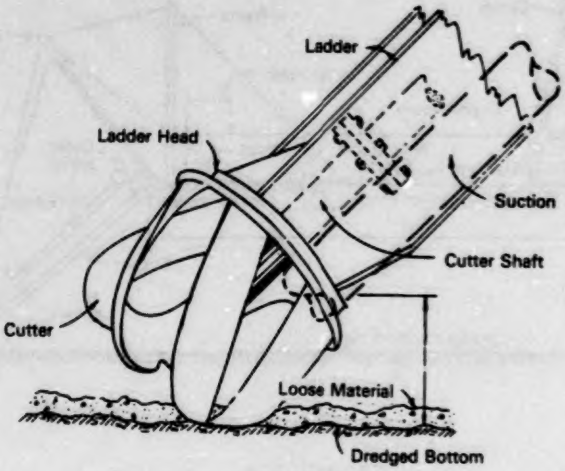

Ladder
Ladder Head
Suction
Cutter Shaft
Cutter
Loose Material
Dredged Bottom

FIGURE 7-7B

STANDARD CUTTER ASSEMBLY: SPIRAL BASKET CUTTER

(Source: Linsley and Franzini, 1979
See Copyright Notice, Page 497

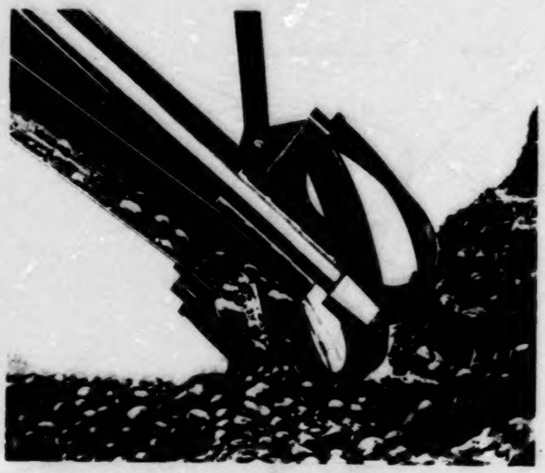


FIGURE 7-8

THE STANDARD CUTTERHEAD DREDGE VESSEL

(Source: Huston, 1976)

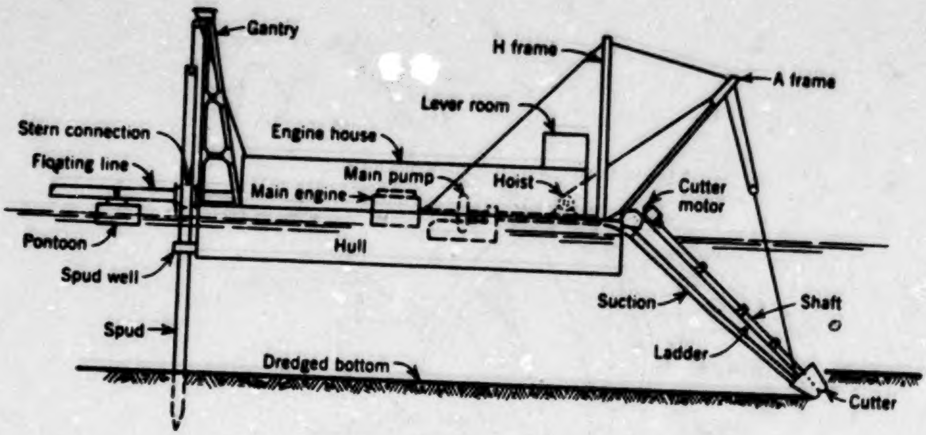

Gantry
H frame
A frame
Lever room
Stern connection
Engine house
Floating line
Main pump
Hoist
Cutter motor
Main engine
Pontoon
Hull
Spud well
Suction
Shaft
Spud
Ladder
Dredged bottom
Cutter

FIGURE 7-9

STANDARD CUTTERHEAD DREDGE OPERATION

(Source: Linsley and Franzini, 1979)
See Copyright Notice, Page 497

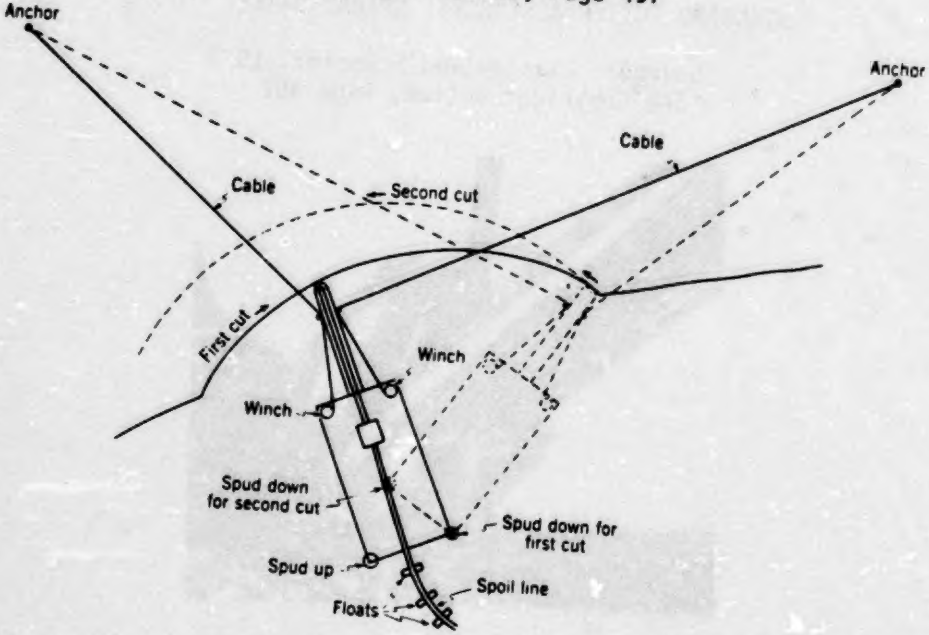
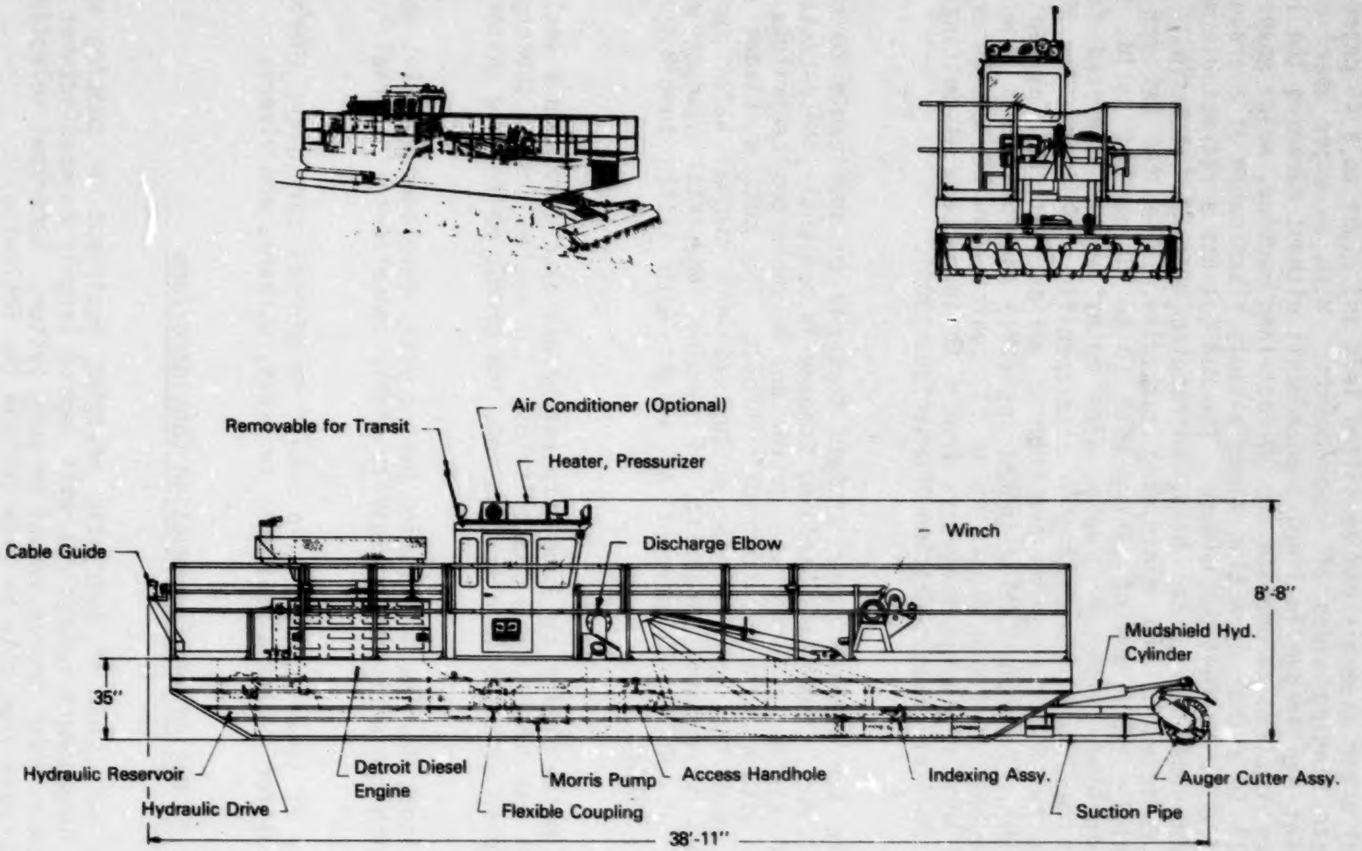

Anchor
Anchor
Cable
Cable
Second cut
First cut
Winch
Winch
Spud down for second cut
Spud down for first cut
Spud up
Spoil line
Floats

FIGURE 7-10

VIEWS OF THE MUD CAT MC-915 DREDGE

(Source: National Car Rental Systems, 1979)


Air Conditioner (Optional)
Removable for Transit
Heater, Pressurizer
Cable Guide
Discharge Elbow
Winch
8'-8"
Mudshield Hyd. Cylinder
35"
Hydraulic Reservoir
Hydraulic Drive
Detroit Diesel Engine
Morris Pump
Flexible Coupling
Access Handhole
Indexing Assy.
Suction Pipe
Auger Cutter Assy.
38'-11"

inches deep, at depths as great as 15 feet and as shallow as 21 inches. The horizontal auger assembly can be tilted left and right to a 45 degree angle to accommodate sloping sides of impoundments. With an auger wheel attachment, the Mud Cat can dredge in lined impoundments without damaging the liner. Two people are required to operate the 30-foot-long machine, which moves by winching itself in either direction along a taut, fixed cable at average operating speeds of 8 to 12 feet per minute. The Mud Cat has a retractable mud shield, which surrounds the cutter head, entrapping suspended material, increasing suction efficiency, and minimizing turbidity. The Mud Cat can discharge approximately 1,500 gpm of slurry with 10 to 30 percent solids through an 8-inch pipeline, although greater flow rates may be achieved for loosely compacted materials. Depending on site-specific conditions, the Mud Cat can remove up to 120 cubic yards per hour of solids (National Car Rental System, Inc., 1980). Vaughan-Maitlen Industries (VMI) manufactures a line of similar "mini dredges" that can remove up to 133 cubic yards of sedimentary material per hour with discharge diameters from 6 to 10 inches, at cutting depths as great as 20 feet (Vaughn-Maitlen Industries, 1980).

There are several other dredges that may be applicable to surface impoundment work. Waterless Dredging Company is presently field-testing a newly developed system in which the cutter and a submerged centrifugal pump are enclosed within a half-cylindrical shroud. The cutting blades remove the material near the front of the cutterhead with minimal water pick-up. The system has a reported capability of pumping industrial sludges with solids contents of 30 to 50 percent by weight with little turbidity generated (Barnard, 1978).

The Delta Dredge and Pump Corporation has also developed a small portable unit that has high solids capabilities. The system uses a submerged 12-inch pump coupled with two counter-rotating, low speed, reversible cutters.

Ellicott Machine Corporation has recently developed a bucket wheel cutter head, which can efficiently excavate highly consolidated material.

Many of these dredges also result in minimal turbidity; therefore, they are useful for dredging sediments in lakes, streams, and rivers.

7.2.2 Design and Construction Considerations

The selection of hydraulic dredging equipment or pumping systems for surface impoundment operations will depend largely on manufacturer specifications for a given dredge vessel or pump system. Important selection criteria that will vary from site to site include the following:

- Maximum depth of the impoundment

- Surface area of impoundment
- Physical nature of material being dredged - e.g., consolidated sludge or hard clay vs. loose sand and gravel
- Chemical nature of dredged material - may dictate special handling procedures; special pipeline for corrosives
- Total volume of material to be dredged
- Distance over which material is to be pumped - proximity of spoil disposal site or treatment facilities
- Terminal elevation of discharge pipeline - contributes to total head to be overcome by pumping; may necessitate use of booster pumps
- Type and amount of aquatic vegetation or overgrowth in impoundment - tree stumps may require special excavation; special cutting attachments for heavy weed growth
- Presence of bottom liner in impoundment
- Power source for dredge or pump systems; availability of electric current
- Ease of access to impoundment
- Maximum size and weight limits for overland transportation
- Cost consideration - see Section 7.2.4.

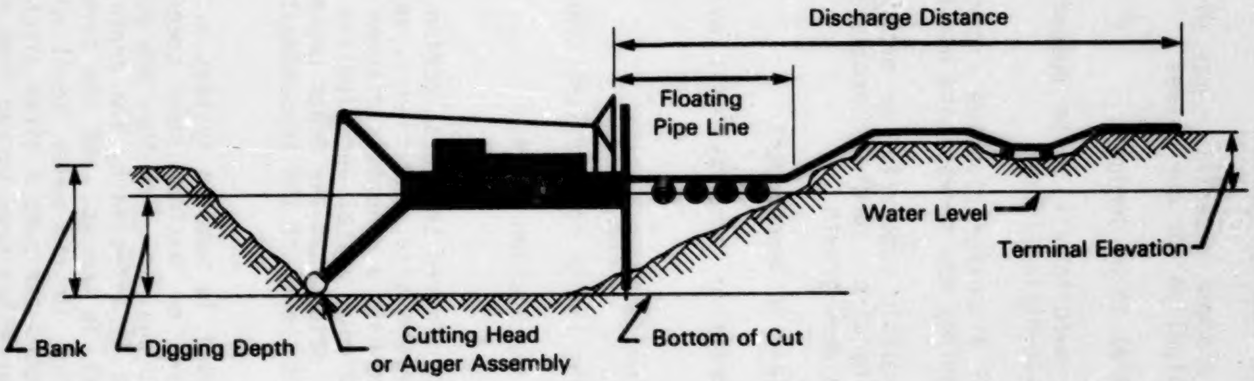
All these criteria must be considered before selection of a pumping system or dredge vessel of the appropriate size, efficiency, and overall capabilities can be made. Figure 7-11 presents a schematic diagram of important distances used in the selection of a dredge vessel and pipeline equipment for a given dredging site. It should be noted that the depth capability of a unit may be increased by lowering the water level of the impoundment.

The centrifugal pumps used in pumping systems or dredge vessels have a rated discharge capacity based on maximum pump speed (in revolutions per minute, rpm) and a given head against which they are pumping. The total head against which pumps must work is affected by the depth of dredging, the distance over which the material is pumped, and the terminal elevation of the discharge pipeline in relation to the water level within the impoundment. This means that centrifugal pumps can pump a given discharge over only a given distance once the dredged slurry has been sucked from the impoundment bottom. If treatment facilities or spoil disposal areas are located at distances or elevations greater than this maixmum pumping capability, then one or more booster pumps (usually centrifugal pumps also) must be installed in the pipeline to pump the slurry to the desired location. In general, for pumping from

FIGURE 7-11

IMPORTANT DISTANCES IN THE SELECTION OF DREDGING SYSTEMS EQUIPMENT

(Source: Ellicott Machine Corp., 1968)


Discharge Distance
Floating Pipe Line
Water Level
Terminal Elevation
Bank
Digging Depth
Cutting Head or Auger Assembly
Bottom of Cut

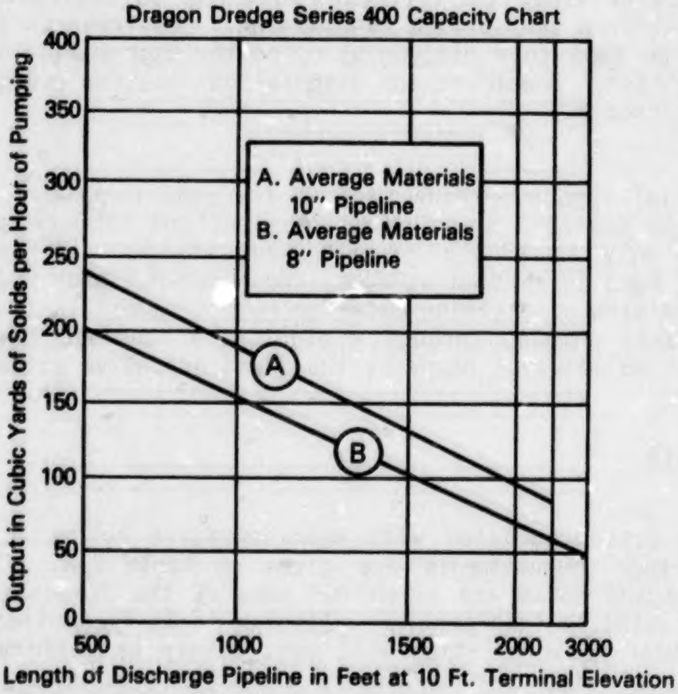
most portable cutterhead dredge vessels, if the disposal or treatment site is located at a distance greater than a half mile away over level terrain, then booster pumps will be required to extend the pumping distance.

The rated discharge capacity of the dredge pump can be used to estimate the solids output (in cubic yards per hour) of the dredging operation, based on the discharge distance, the size of the discharge pipeline, and the solids content (percent by volume) of the pumped slurry--generally on the order of 10 to 30 percent. Figure 7-12 shows the capacity chart for Ellicott Machine's

FIGURE 7-12

TYPICAL CAPACITY CHART FOR PORTABLE CUTTERHEAD DREDGE

(Source: Ellicott Machine Corp., 1969)



Series 400 "Dragon" dredge, which relates solids output (in cubic yards per hour) to the length of the discharge pipeline at a given terminal elevation. Such a chart can be developed for a given dredging operation and used to predict dredged material production over time. Keep in mind, however, that dredging output may be affected by a number of uncontrolled site-specific variables, including weather conditions, local topography, and unforeseen equipment failures or dredging obstacles. These will influence both performance and cost of the dredging operation.

7.2.3 Advantages and Disadvantages

Advantages and disadvantages of hydraulic dredging of surface impoundments are summarized in Table 7-7. The main disadvantage associated with hydraulic removal of materials from surface impoundments is the necessity of locating and/or constructing dewatering/disposal areas (or treatment facilities) within economical distances of the dredging site. Spoil containment facilities must be able to handle large volumes of dredged material in a liquid slurry form, unless dewatering is performed prior to spoil transport (which will entail added costs also). Processing equipment to dewater, segregate, or chemically treat the dredged slurry may be desirable prior to land disposal of spoil from the surface impoundment. Requirements for dredge spoil dewatering can be kept to a minimum by using the high-solids dredging systems previously discussed. Treatment and disposal options for dredged slurries are addressed in Section 9.3.

Where special lagoon pumping systems are used to dredge impounded materials at in-situ density, spoil dewatering may not be necessary. But these are applicable only to highly consolidated, viscous materials with a high solids content, such as settled sludges. Most conventional hydraulic dredging involves the addition of dilution water to bottom materials to form a pumpable mixture that makes pumping through a pipeline a feasible transportation alternative; it is not always, however, the least expensive alternative.

7.2.4 Costs

The unit costs associated with representative hydraulic dredging techniques for surface impoundments are given in Table 7-8. Capital purchase costs and operating costs are given for some of the dredge vessels (and accessories) discussed in this section, although it is recognized that hydraulic dredging for local municipalities will most likely be performed by specialty contractors whose rates may be highly variable from site to site. If a dredging contractor is to be used, costs of dredging can be estimated at $3-5 per cubic yard of material removed.

TABLE 7-7

SUMMARY OF EVALUATION OF HYDRAULIC DREDGING OF SURFACE IMPOUNDMENTS

Advantages	Disadvantages
● Efficient removal of solids/water mixtures from impoundments	● Necessity of locating spoil management facilities (dewatering; disposal; treatment) nearby
● Removes hazardous materials in readily processed form (slurry)	● Necessitates high volume handling of solids/water mixtures
● Suitable for removal of materials from surface impoundments in wide range of consistencies--from free-flowing liquids to consolidated/solidified sludges	● May require booster pumps for long-distance transport of dredged slurries
● Utilizes well-established, widely available technology	● Mobilization and demobilization may be time-consuming and costly
	● Cannot remove large items (such as drums)

Other cost considerations that are not included in Table 7-8 are the following:

- Crane rental to launch and retrieve portable dredge vessels
- Freight and handling costs for shipping dredge equipment
- Transportation of equipment from site to site
- Insurance (hull coverage and liability) for purchased vessels
- Storage and/or warehouse costs
- Sales tax

TABLE 7-8

UNIT COSTS FOR HYDRAULIC DREDGING OF SURFACE IMPOUNDMENTS

Description	Unit cost	Source
Vaughan Model 335 Lagoon Pumper, with 100 hp motor and 6-inch chopper pump; complete platform assembly including winch (shipping weight approx. 6,500 lbs)	$27,400 each	1
Mini-Mud Cat SP-810 with 10-foot boom and submerged pump; less accessories (12,200 lbs)	$68,720 each	2
● weed auger assembly	$1,680	2
● auger wheels (for lined ponds)	$1,347	2
● discharge pipe package, 1,500 feet of 6-inch polyethylene pipe (10,106 lbs)	$18,811	2
● cable and harnessing equipment (1,656 lbs)	$5,374	2
● extra sections of carrier pipe, 6-inch x 19 feet (56 lbs)	$126.20	2
Mud Cat MC-915 with 15-foot boom; less accessories (21,000 lbs)	$106,313 each	2
● air conditioned cab	$1,605	2
● weed auger assembly	$1,712	2
● Ni-hard pump	$2,502	2
● auger wheels (for lined ponds)	$1,766	2
● right angle cutting knives, extra	$222	2
● recessed impeller pump (for heavy weed or debris applications)	$1,766	2
● turbocharged engine (high altitude operation)	$1,926	2

--continued--

TABLE 7-8 (Continued)

Description	Unit cost	Source
● discharge pipeline package, 1,500 feet of 8-inch polyethylene pipe with PVC floats (8,911 lbs)	$20,629	2
● cable and related harnessing equipment (995 lbs)	$3,524	2
● extra sections of polyethylene carrier pipe, 8-inch x 19 feet, including couplings and gasket with float bands and links (85 lbs)	$175.50	2
● booster pump, skid mounted, with connector fittings (7,977 lbs)	$31,246	2
● service boat and motor (466 lbs)	$1,774	2
Lease (towards purchase) of Mud Cat MC-915 with discharge pipeline package and cable equipment;		
first two months	$11,000/month	2
third month +	$ 8,300/month	2
Operating costs for Mud Cat		
● Fuel - 6.5 gal/hr x $1.00/gal	$ 6.50/hour	2
● lubricants	$ 1.00/hour	2
● repairs (parts and labor)	$ 4.25/hour	2
● two operators	$20.00/hour	2
Total hourly operating costs	$31.75/hour	2
VMI Mini Dredge 815 with standard equipment (16,000 lbs)	$79,500 each	3
● weed cutting attachment	$895	3
● cable and harness	$3,985	3
● 10-inch PVC carrier pipe	$6.22/foot	3
● floats for carrier pipe	$2.00/foot	3
● 10-inch flexible industrial tubing	$55.70/50-ft	3
● service boat with motor	$1,325	

--continued--

TABLE 7-8 (Continued)

Description	Unit cost	Source
Contractor hydraulic dredging (suction or cutterhead), materials pumped 1,000 feet to shore dump (for inland rivers in South, deduct 30%)	$\$3\text{-}5/\text{yd}^3$	4

---

[1] Vaughan Co., Inc., 1980.
[2] National Car Rental Systems, Inc., 1980.
[3] Vaughn-Maitlen Industries, 1979.
[4] Godfrey, 1979.

7.3. LAND DISPOSAL

7.3.1 General Description and Applications

Land disposal is the most commonly practiced method for disposal of industrial and municipal wastes. It includes disposal in secured landfills, surface impoundments, and land application.

The use of secured landfills for the disposal of hazardous waste may continue for some time since other ultimate disposal techniques are not yet well developed. Such landfills are typically constructed with impermeable natural subsoils or man-made liners that inhibit the movement of leachate to ground and surface waters. In some cases, impermeable covers are required to reduce the mobilization of leachate and to reduce the possibility of contamination of surface water runoff. Monitoring, nevertheless, is required in ground and surface waters around the landfill and this must be continued on a regular basis in order to provide a gage on the integrity of the liner and leachate collection system. Because of the notoriety given to improperly designed landfills or impoundments in the past, the institutional (as well as political) implications involved in landfill and impoundment siting may be more time-consuming and difficult than the environmental and engineering aspects of an acceptable facility.

In some cases, application of hazardous wastes on the land may be feasible. This involves the uniform application of selected wastes to a specific area and includes mixing with upper soil layers. This is usually done after the wastes have been converted to environmentally acceptable materials by means of biological, chemical, or physical treatment. This technique has been used for the disposal of municipal sludges and petroleum industry wastes. For those wastes that are resistant to biodegradation, the potential of contamination of surface water runoff and infiltration exists. In addition, uptake of unaltered materials by vegetation can introduce potentially hazardous compounds into the food chain.

7.3.2 Design and Considerations

Subtitle C of the Resource Conservation and Recovery Act (RCRA) provides authority to EPA to promulgate regulations designed to control hazardous waste disposal. Parts 264 and 265 in the Federal Register, Volume 45, No. 98 (May 19, 1980) are the first phase of EPA's requirements under Section 3004 of RCRA for owners and operators of hazardous waste disposal sites. The requirements under Section 264 and 265 are, in some respects, purposely general; EPA's position is that much additional research is needed to provide more specific guidance to landfill operators, and such research is currently underway. Section 264 establishes minimum national standards that define acceptable management of hazardous wastes. Section 265 establishes requirements applicable during interim status or that period after application for a permit but before final administrative action on the permit. With the huge number of applications likely to flood EPA, it may be several years before final administrative action on all permits is taken. Therefore, landfill owners and operators will need to thoroughly familarize themselves with the requirements of Parts 264 and 265.

Owners with interim status are required to meet the following criteria as well as others specified in Parts 265 and 265. Parts 264 and 265 must be consulted for additional detail.

- Owners must develop and follow a waste analysis plan that specifies tests to be used and their frequency in order to determine the identity of incoming wastes.
- Groundwater monitoring is required at all landfills, impoundments, and landfarms. Landfarms require, in addition, an unsaturated zone monitoring system. Site security requires a 24-hour surveillance system that monitors and controls entry, an artificial or natural barrier surrounding the site, and measures to control entry at all times.
- Owners or operators must develop and follow a written schedule for inspection of monitoring equipment, safety and emergency equipment, and operating and structural equipment. The rate of inspection is based on equipment deterioration and the probability of a human health hazard with failure.

- Monitoring and maintenance will be required for a post-closure period of 30 years.
- Containers are required to be compatible with the wastes. They must be inspected on a weekly basis and if leaks are occurring, the wastes must be transferred to other containers or otherwise properly managed.

Part 265 also establishes some specific requirements for landfills and surface impoundments.

Runon to landfills must be diverted and runoff must be collected. Wind dispersal of hazardous wastes from landfills must be controlled. Bulk or non-containerized hazardous liquid wastes must not be placed in a landfill unless there is a compatible liner and a leachate collection system or the liquid waste is treated or stabilized chemically or physically.

For surface impoundments, Part 265 requires that there be sufficient freeboard to prevent overtopping of the dike by overfilling, wave action, or storms and that it be at least 2 feet. All earthen dikes are required to have protective cover such as grass, shale, or rock to minimize wind and water erosion. Specific slopes of dikes have been recommended and are such as to insure the integrity of the dike.

Part 265 of RCRA also requires certain monitoring and maintenance activities after closure of a site that has been used to dispose of hazardous waste. In particular, EPA requires a 30-year leachate monitoring program after the site has ceased to accept hazardous wastes, although this requirement could be lengthened or shortened by review on a case-by-case basis. The most significant (and probably the most difficult) aspect of closure involves the funds necessary to maintain and to monitor the facility over an extended period of time. Trust funds, expenses drawn out of operating revenues, and private and public insurance programs have all been considered, but none has permitted the designation of permanent, specific guidelines. As a result, EPA has set up interim guidelines to allow administration of land disposal sites to proceed. These guidelines include:

- Notice in Deed to Property - a recording of instructions in the use of the property after its resale, to ensure that liner or cover integrity be maintained.
- Submission of Closure Plans - to assure that an operating facility has the proper measures to deal with closure and is building up (or has adequate) finances for post-closure activities.
- Time Allotted for Closure - in order to effect closure within a six-month to three-year interval after operations have ceased, depending on a review on a case-by-case basis.

- Post Closure Permits - to require owners or operators for closed facilities to provide a program of access, monitoring, and financial responsibility.

7.3.3 Advantages and Disadvantages of Land Disposal Methods

Land disposal has generally been the most expedient, economical, and best understood method of disposing of wastes. RCRA requirements will significantly increase the cost of land disposal by requiring more stringent site security, monitoring, and long-term liability and management. There will be increased incentive for using waste destruction or recovery techniques that do not require long-term management and liability.

Nevertheless, land disposal in a site that meets RCRA and state requirements will continue to be a viable disposal method, particularly until alternative technologies are fully developed and facilities are sufficiently prevalent to handle much of the waste currently disposed of by land.

7.3.4 Costs

Costs for hauling of wastes are estimated to range from $2-4 per loaded mile. Disposal costs approximate $240/ton for very hazardous material, $120/ton for flammable wastes, $85/ton for most industrial sludges, and $40-50/ton for municipal treatment sludges (Cecos International, 1980).

7.4 SOLIDIFICATION

Waste solidification involves a number of techniques designed to seal the wastes in a hard, stable mass. Nearly all of these techniques are an outgrowth of radioactive waste disposal and a U.S. Department of Transportation (DOT) regulation forbidding the transportation of liquid radioactive wastes (Pojasek, 1978). Much of the information on solidification techniques is proprietary and therefore not included in this discussion.

Waste solidification practices are limited in their applicability to remedial measures for waste disposal sites. One major reason for this is cost. In addition to the cost of excavating the wastes (see Section 7.1), the wastes must undergo a thorough analytical characterization and often a stabilization process to ensure compatibility with the solidification process. Moreover, these processes are, for the most part, very waste-specific. Since many solidification techniques evolved from the need to transport non-liquid radioactive wastes, not all methods result in a solid with long-term stability. Several methods result in a material that, while easy to handle and transport, is not meant to secure the wastes over a long period of time.

These restrictions limit the use of solidification as a remedy for radioactive or highly toxic wastes.

It should be noted that many sludges contain significant concentrations of organic chemical and hydrocarbon constituents. Therefore, the organic fixing capacity of the various processes should be carefully reviewed and compared in regard to potential leachates of such pollutants to form the solidified mass.

Solidification methods can be placed in two main groups based on how the wastes are held in the solid. Some methods physically surround the waste particles with the solidifying agent. Other methods chemically fix the wastes in a reaction with the solidifier. Included in these two broad categories are the six most commonly used methods:

- Cement-based
- Lime-based
- Thermoplastics
- Organic polymers
- Self-cementation
- Glassification

Each of these methods is discussed below:

7.4.1 Cement-Based Solidification

7.4.1.1. General Description

This method involves sealing the wastes in a matrix of Portland cement, a very common construction material. Most solidication is done with Type I Portland cement but Types II and V can be used for sulfate or sulfite wastes (EPA, 1979b). This method physically or chemically solidifies the wastes, depending upon waste characteristics.

7.4.1.2 Applications

Cement-based solidification techniques are among the most versatile. They can neutralize and seal acids and can handle strong oxidizers such as chlorates and nitrates (EPA, 1979b). These methods are also good for solidifying many toxic metals, since at the pH of the cement (pH 9-11), many metals are insoluble carbonates and hydroxides. Certain wastes can cause problems with the set, cure, and permanence of the cement waste solid unless the wastes are pretreated. Some of these incompatible wastes are:

- Sodium salts of arsenate, borate, phosphate, iodate, and sulfide
- Salts of magnesium, tin, zinc, copper, and lead
- Organic matter
- Some silts
- Some clays
- Coal or lignite (EPA, 1979b)

7.4.1.3 Special Considerations

A typical cement-based solidification process results in wastes that are twice the weight and volume of the original. Basically, this means that reburial of the solidified wastes would require twice as large an area.

7.4.1.4 Advantages and Disadvantages

Provided this process is used on compatible wastes, the short-term effectiveness can be expected to be quite good. The equipment for cement mixing is commonplace and the process is quite tolerant of chemical variations. However, because cement is a porous solid, contaminants can be leached out of the matrix over time and is, therefore, usually not effective for organic wastes. Although it is possible to seal the outside of a block of cement-solidified wastes using styrene, vinyl, or asphalt to prevent leaching, no commercial systems are available to do this (EPA, 1979b).

7.4.1.5 Costs

Cement costs range from $58 to $80 per ton at the mill (1982) and cement is added to make a product that is about 130 percent of volume of the original wastes. However, capital expenditure and transportation will vary widely depending on the site and the waste. Cost information for specific wastes should be obtained from vendors. Vendors include: Atcor Washington, Inc., Park Mall, Peeksville, New York; and Chemfix, Inc., Kenner, Louisiana.

7.4.2 Lime-Based Solidification

7.4.2.1 General Description

Lime-based solidification is very similar to cement-based, though the solidifier is a mixture of a fine-grained siliceous material, lime, and water. Other wastes, such as fly ash, cement-kiln dust, or ground blast-furnace slag can be used as the siliceous additive (EPA, 1979b). When used with the right types of wastes, this method results in a fairly stable, easily transported solid.

7.4.3.2 Applications

In order to be compatible with lime-based solidification, wastes should be stable at high pH. The most common use of this technique is to solidify flue gas cleaning sludge using fly ash from the same plant, along with lime and other additives (EPA, 1979b).

7.4.2.3 Special Considerations

As with cement solidification, the lime-based method increases the weight and volume of the wastes. The amount of overall increase is related to the nature of the original wastes. If, for instance, waste fly ash is added as the solidifying agent rather than some nonwaste additive, two types of wastes are included in a solid of the same size and weight.

7.4.2.4 Advantages and Disadvantages

Major advantages of this technique include the ready availability and low cost of materials and the familiarity of commonly used equipment. A disadvantage is that the solid mass resulting from lime-based solidification is porous. As such, it must either be sealed or placed in a secure landfill to prevent leaching of contained wastes. Another major disadvantage is that sludge or wastes containing organics cannot be treated.

7.4.2.5 Costs

Cost of lime fixation should be determined on a case-by-case basis. Overall costs have been reported to range from $0.03 to $0.15 per gallon of industrial sludge (Palesh and Gulledge, 1979).

7.4.3 Themoplastic Solidification

7.4.3.1 General Description

Thermoplastic solidification involves sealing wastes in a matrix such as asphalt bitumen, paraffin, or polythylene. These substances are solid at normal temperatures, but liquid when heated.

7.4.3.2 Applications

Thermoplastic compounds should not be considered for solidifying:

- Organic solvents
- Iron and aluminum salts
- Strong oxidizers
- Anhydrous salts
- Wastes that may break down when heated (EPA, 1979b)

7.4.3.3 Special Considerations

Thermoplastic solidification requires specialty equipment and highly trained operators to heat and mix the wastes and solidifier. The common range of operating temperatures is 130° to 230° (EPA, 1979b). The energy intensity of the operation is increased by the requirement that the wastes be thoroughly dried before solidification. Certain wastes, such as tetraborates, and iron and aluminum salts can cause premature solidification, and plug up the mixing machinery (EPA, 1979b). Some thermoplastic/waste mixtures cure to a very plastic solid and require some secondary containment, such as a 55-gallon drum, for handling and disposal.

7.4.3.4 Advantages and Disadvantages

Thermoplastically solidified wastes have been shown to lose contaminants through leaching at a far slower rate than either cement or lime-solidified wastes. Also, these thermoplastics are little affected by either water or microbial attack (EPA, 1979b). In addition, this method may permit some recovery, may effectively handle some organics, and can reduce landfill volume. These factors combine to give this method excellent long-term effectiveness. Principal disadvantages include the high cost of equipment and high energy utilization.

7.4.3.5 Costs

Capital and operating costs are generally not available for non-radioactive disposal operations. Vendors should be consulted for specific cost information. One such vendor is: Werner and Pfleiderer Corp., Waldwick, New Jersey.

7.4.4 Organic Polymer Solidification

7.4.4.1 General Description

The primary organic polymer use for solidification is urea-formaldehyde, but others include vinyl ester-styrene and polyester. This physical solidification method involves mixing the wastes with prepolymers, polymers, and a catalyst to form the solid mass (EPA, 1979b). In some cases, it is possible to mix the wastes and the polymers in a 55-gallon drum, add the catalyst, and have solidification take place in a convenient disposal container.

7.4.4.2 Applications

Since organic polymers do not enter into reactions with the wastes, a somewhat wider range of wastes can be solidified than with most other methods. With the most common solidification polymer, urea-formaldehyde, however, the wastes should be stable at low pH, since the catalyst used is strongly acidic (EPA, 1979b). Acid-catalyzed polymerization is generally highly exothermic. Waste encapsulated in inorganic polymers should be devoid of thermally unstable or high vapor pressure toxicants that might be emitted to ambient air.

In solidifying wastes with organic polymers, the wastes do not have to be dried prior to polymerization. Any liquid associated with the waste will remain after polymerization, if not dried, and the polymer mass must often be dried before disposal. Significant amounts of residual water may have an effect on the physical integrity of the polymer structure.

7.4.4.3 Special Considerations

As noted earlier, the catalyst used with urea-formaldehyde polymer is highly acidic and thus very corrosive. This means that the equipment used to mix and contain these substances must be made of a corrosion-resistant material.

Another point that illustrates the importance of proper waste/polymer drying is the tendency of the solid to "weep" or release any uncombined water. The weep water is strongly acidic and often laden with pollutants. Secondary containment as in a lined 55-gallon drum, can help to control this problem.

7.4.4.4 Advantages and Disadvanges

The long-term effectiveness of the various organic polymers is questionable. Many of the polymers, and especially urea-formaldehyde, are biodegradable, and so could readily release any contained pollutants. This fact alone shows that these solidification methods are more for ease of handling than for security of the wastes. Hazardous or harmful releases may occur from catalysts, "weep" water, and fumes from the resins. One advantage of this process is that the resin formed is less dense than cement, and transportation costs are significantly reduced. Little is known about suitable waste types for some polymer solidification methods.

7.4.4.5 Costs

Since little testing has been done on non-radioactive wastes, cost data are not widely available. Costs for nuclear wastes have been reported at $2.75 per gallon (Palesh and Gulledge, 1979). Todd Shipyard Corp., Galveston, Texas markets an organic polymer called "Safe-T-Set." This vendor and others should be contacted for specific cost data.

7.4.5 Self-Cementing Solidification

7.4.5.1 General Description and Applications

This process is specific to certain types of wastes, and is similar to cement-based solidification in that a portion of the wastes (8-10%) are calcined to form a cement. Wastes that can be considered for solidification by this method are flue gas cleaning or desulfurization sludges that contain a large amount of calcium sulfate or sulfite (EPA, 1979b) and ion exchanger regenerant wastes from the utilities industry, since these contain appreciable amounts of calcium sulfate and sodium sulfate.

7.4.5.2 Special Considerations

The requirement that a portion of the wastes be calcined into cement necessitates special equipment and specially trained workers to carry out this process. In addition, the calcining step requires an increased energy input.

7.4.5.3 Advantages and Disadvantages

Self-cementation of cleaning sludges produces a solid that retains metals and is stable, non-flammable, and non-biodegradable. However, the process is limited in applicability in that only high sulfate or sulfite sludges can be used. The process is also energy-intensive and requires specialized equipment (EPA, 1979b).

7.4.5.4 Costs

Costs of self-cementing solidification are $2-2.75/ton of sludge (EPA, 1979b).

7.4.6 Glassification

7.4.6.1 General Description

According to EPA (1979b), glassification of wastes involves combining the wastes with molten glass at a temperature of 1,350°C or greater. However, the encapsulation might be done at temperatures significantly below 1,350°C (a simple glass polymer such as boric acid can be poured at 850°C). This melt is then cooled into a stable, non-crystalline solid.

7.4.6.2 Applications

This process is quite costly and so has been restricted to radioactive or very highly toxic wastes (EPA, 1979b). To be considered for glassification, the wastes should be either stable or totally destroyed at the process working temperature.

7.4.6.3 Special Considerations

Glassification of wastes is an extremely energy-intensive operation, and requires sophisticated machinery and highly trained personnel.

7.4.6.4. Advantages and Disadvantages

Of all the common solidification methods, glassification offers the greatest degree of containment. Most resultant solids have an extremely low leach rate. Some glasses, however, such as borate-based glasses, have high leach rates and exhibit some water solubility. The high energy demand and requirements for specialized equipment have limited its use.

7.4.6.5 Costs

No information.

7.5. ENCAPSULATION

7.5.1 Description and Applications

Encapsulation is the process by which hazardous wastes are physically enclosed by a synthetic encasement to facilitate environmentally sound transport, storage, and disposal of the wastes. As a remedial action, encapsulation may be used to seal particularly toxic or corrosive hazardous wastes that have been removed from disposal sites. Waste types that may require encapsulation include the following:

- Solid hazardous wastes in bulk or particulate form (e.g., severely contaminated sediments)
- Dewatered hazardous sludges
- Containerized hazardous wastes (solids, sludge, or liquid) in damaged or corroded drums
- Hazardous wastes which that been stabilized through solidification/cementation (by processes described in Section 7.7)

TRW Systems Group has successfully developed bench-scale processes to (1) agglomerate and encapsulate toxic and corrosive heavy metal sludges and soluble heavy metal salts; and (2) encapsulate containerized wastes. The agglomeration/encapsulation process involves mixing dried sludges (containing such hazardous heavy metals as arsenic, lead, mercury, selenium, beryllium, cadmium, zinc, and chromium) with a binder resin (modified 1,2-polybutadiene) and thermosetting the mixture in a special mold, while applying moderate mechanical pressure. The agglomerated material is a hard, tough solid block. Encapsulating the waste/binder agglomerate with a 1/4-inch seamless jacket of high density polyethylene (HDPE) is accomplished by packing powdered polyethylene around the block and then fusing the powder in-situ with a second metal sleeve mold. A schematic diagram of the apparatus used to encapsulate the agglomerate is shown in Figure 7-13. Figure 7-14 summarizes the agglomeration/encapsulation process flow. A commercial-scale encapsulate produced by this method is expected to be a solid cube, 2 feet on edge, weighing 800 to 1,000 pounds. It would require approximately 8 percent (by weight) of polybutadiene resin for its fabrication (Lubowitz and Wiles, 1978). Presumably, as the technology advances, additional jacket sizes will be available.

The second TRW encapsulation process is designed to enclose and seal waste containers such as 55-gallon drums (subject to corrosion, rupture, leaks, and spills) using the same basic mold and fusion apparatus. To provide load-bearing ability, a 1/8-inch-thick interior casing of fiberglass is used to reinforce the 1/4-inch-thick HDPE jacket that encapsulates the container. A commercial-scale, fiberglass-reinforced HDPE encapsulate is envisioned to provide up to 75 gallons of capacity. The cylindrical jacket and casing would comprise about 5.3 percent (by volume) of the total encapsulate volume. Commercially, 1/4-inch-thick HDPE jackets can be fabricated in 30 seconds (Lubowitz and Wiles, 1978).

Comprehensive laboratory testing of bench-scale encapsulates has demonstrated their ability to withstand severe mechanical stresses and biological and chemical degradation. Encapsulates containing wastes of various solubility were exposed to leaching solutions of various corrosivity; results indicate that the encapsulated wastes were completely isolated from, and resistant to, simulated disposal environment stresses. The encapsulates were also found extremely resistant to mechanical deformation and rupture. They exhibit high compressive strength and outstanding ability to withstand impact, puncture, and freeze-thaw stresses (Lubowitz et al., 1977).

7.5.2 Design and Construction Considerations

It is important to emphasize that encapsulation techniques have only recently advanced from the developmental and testing stages, and no large commercial-scale encapsulation facilities have been designed and operated as yet. It is likely that, as a remedial action, encapsulation will not be an economically feasible alternative to other direct waste treatment methods. However, a central solidification/encapsulating waste processing facility may be technically and economically feasible as a pre-disposal operation at hazardous waste storage and disposal facilities in the near future.

The fabrication of commercial-scale encapsulates of containerized wastes under actual field conditions would require an encapsulation unit that is readily transportable to the storage or disposal site where containerized wastes reside. Where containerized wastes are of volumes smaller than the design capacity of the encapsulation unit, sand or soil may be used to fill voids between the container and encapsulate walls. Where very large volume waste containers require encapsulation (greater than 55 gallons), it may be necessary to install compaction operations at the site (Lubowitz and Wiles, 1978).

FIGURE 7-13

ENCAPSULATION PROCESS CONCEPT

(Source: Lubowitz et al., 1977)

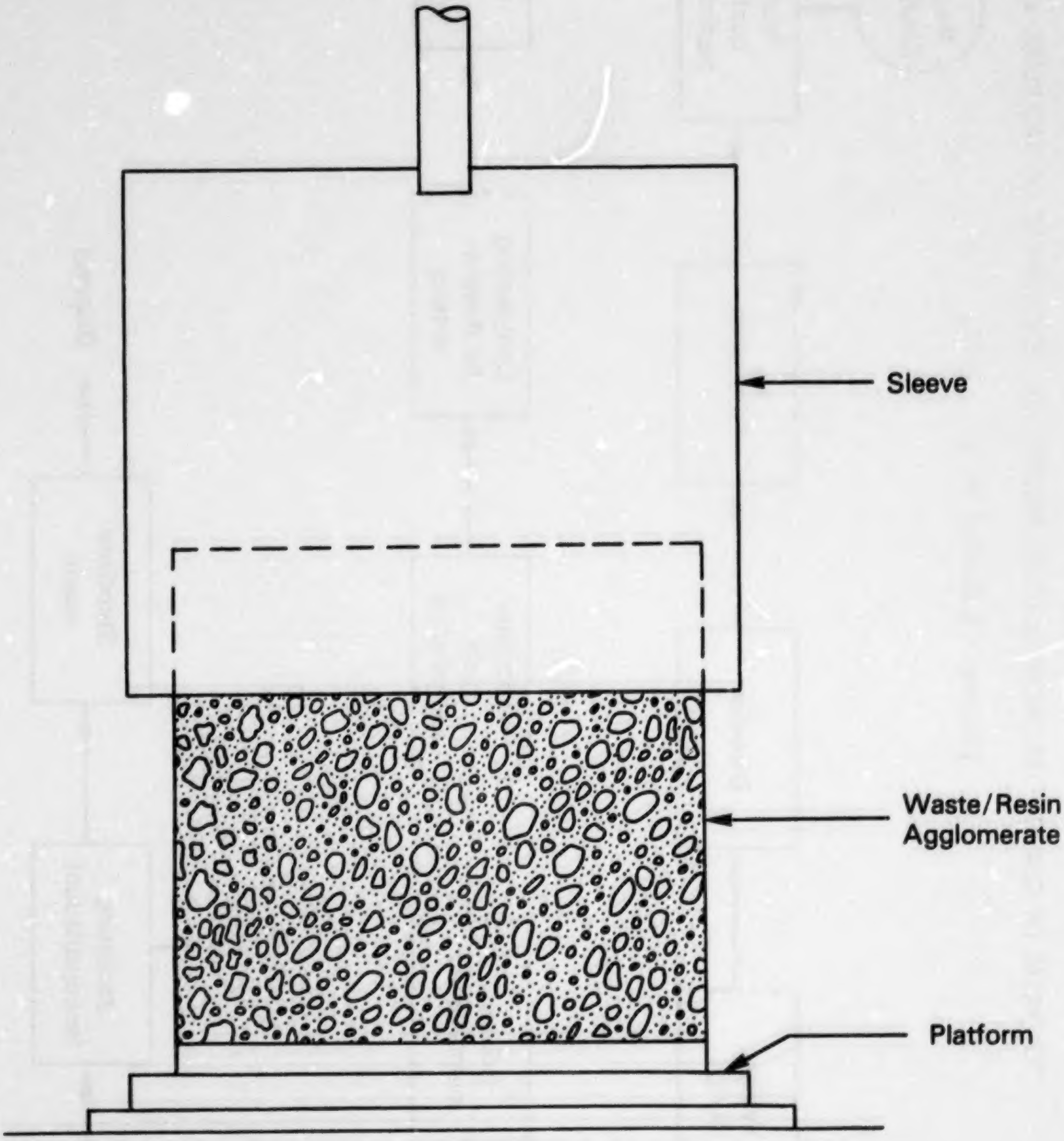
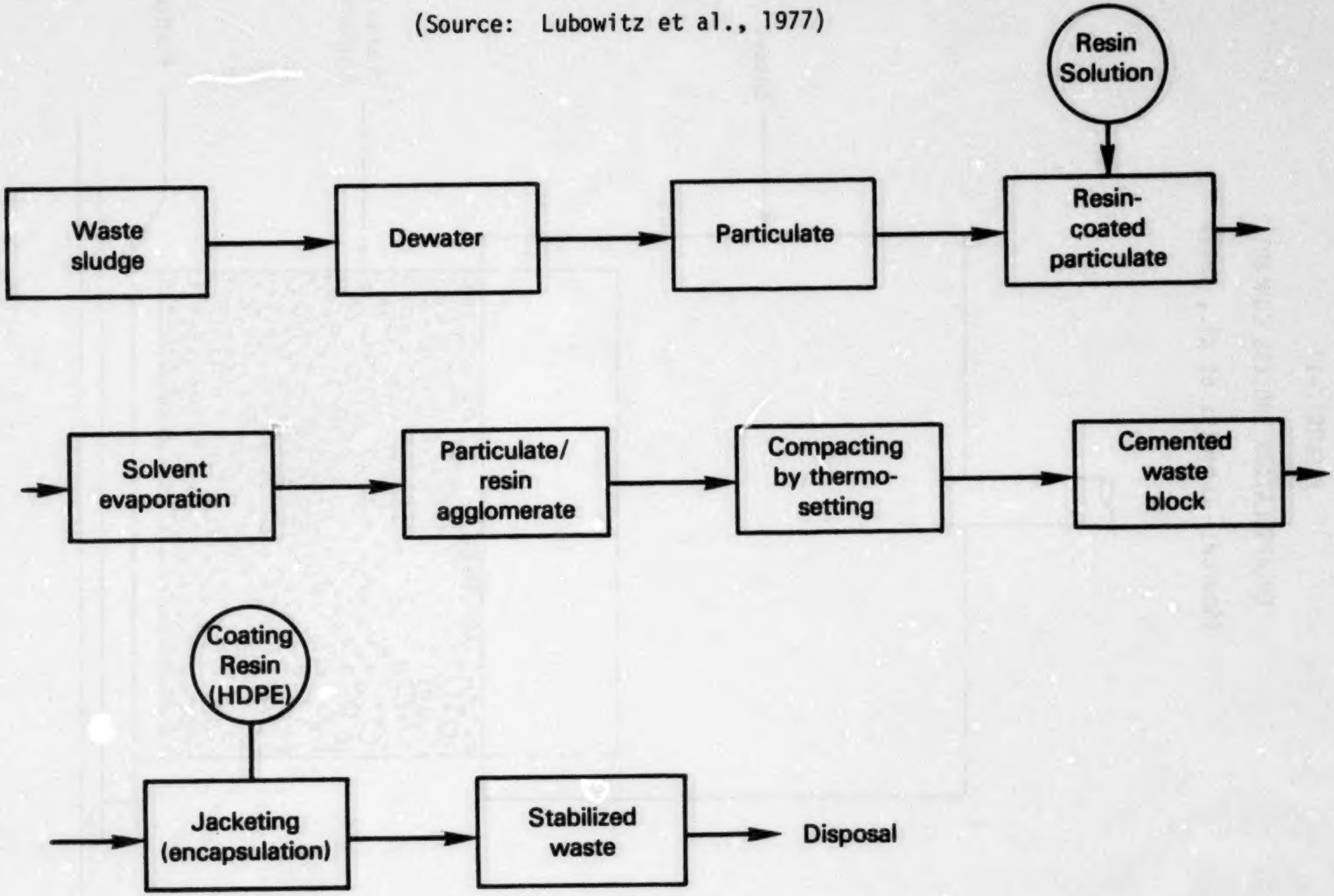

Sleeve
Waste/Resin Agglomerate
Platform

FIGURE 7-14

SCHEMATIC OF CEMENTATION/ENCAPSULATION PROCESS FOR CONTAINMENT OF HAZARDOUS WASTES

(Source: Lubowitz et al., 1977)


Resin Solution
Waste sludge
Dewater
Particulate
Resin-coated particulate
Solvent evaporation
Particulate/resin agglomerate
Compacting by thermo-setting
Cemented waste block
Coating Resin (HDPE)
Jacketing (encapsulation)
Stabilized waste
Disposal

7.5.3 Advantages and Disadvantages

The major advantage of encapsulation processes is that the waste material is completely isolated from leaching solutions, and soluble hazardous materials such as heavy metal ions and toxic salts can be successfully encapsulated. The impervious HDPE jacket eliminates all leaching into contacting water (which may infiltrate or flow over disposal sites) and effectively contains hazardous waste substances that might otherwise migrate offsite. Other advantages associated with hazardous waste encapsulation include:

- The cubic and cylindrical encapsulates allow for efficient space utilization during transport, storage, and disposal
- The hazard of accidental spills during transport is eliminated
- HDPE is low in cost, commercially available, very stable chemically, non-biodegradable, mechanically tough. and flexible
- Encapsulated waste materials can withstand the mechanical and chemical stresses of a wide range of disposal schemes (landfill, deep-well disposal, ocean disposal)

There are, however, major disadvantages associated with encapsulation techniques. Among these are:

- The binding resins required for agglomeration/encapsulation (polybutadiene) are expensive
- The process requires large expenditures of energy in fusing the binder and forming the jacket
- The system requires large capital investments in equipment
- Skilled labor is required to operate molding and fusing equipment
- Drying/dewatering of non-containerized waste sludges is required for agglomeration/encapsulation
- The process has yet to be applied on a commercial scale under actual field conditions

7.5.4 Costs

TRW has developed a process economic model to predict the estimated costs associated with the commercial-scale operation of an agglomeration/ encapsulation facility. By this model, it was determined that a plant with a 20,000 to 60,000 tons per year processing capacity would require an initial capital investment of approximately $1.4 million. This represents the total cost of installed equipment, which includes raw material storage tanks, a rotary tray drier for drying sludges, two agglomerate molds, a coating mold, curing oven, powdered HDPE delivery system, steam boiler, raw material feed pumps, and a conveyor belt system. Such a plant would incur base operating costs of about $1.8 million annually; this translates to a $91 per ton overall processing cost for waste materials encapsulated at the plant (Lubowitz et al., 1977).

Without dewatering, the overall cost of this agglomeration/encapsulation process can be broken down by source as follows: 50 percent, raw material requirements; 25 percent, labor costs; and 25 percent, equipment costs. The cost of commercial agglomerating resins such as polybutadiene account for the major portion of raw material costs; the use of impure, scrape resins (rather than virgin, commercial resins) may substantially reduce the $91 per ton processing cost (Lubowitz et al., 1977). It is also important to note that these costs are derived for the encapsulation process based on initial agglomeration of the waste material. Encapsulation of containerized hazardous wastes using fiberglass-reinforced HDPE jackets does not require initial agglomeration, and does not require the use of expensive polybutadiene resin, and therefore will be much more economically feasible as a direct waste treatment method. Also, the encapsulation can be done by a portable system that can be transported from disposal site to disposal site.

7.6 IN-SITU TREATMENT

An alternative to the removal and subsequent treatment of land-disposed wastes is to treat the wastes in-place, without dredging or excavation. A number of conceptual techniques may be applicable as "in-situ" treatment methods. These techniques may be feasible for sites where wastes are well defined, shallow, and the extent of contamination is small. Such limitations suggest specific applicability to chemical spills; soil contaminated by surface leachate; landspreading operations; dredge spoil containment basins; and small, shallow industrial surface impoundments where the waste has been well characterized. These methods are briefly discussed in the following section.

7.6.1 Solution Mining

Solution mining or soil flushing is the process of flooding the land-disposed waste material or area of contamination with a solvent and collecting the elutriate with a series of shallow well points. The elutriation process is based on the concept of mobilizing the contaminant(s) into the solvent phase via solubility or chemical reaction. The most probable, feasible, and cost-effective techniques use water as the base solvent.

Water may be the only material used if the waste or contaminant is readily soluble. Solutions of sulfuric, hydrochloric, nitric, phosphoric, and carbonic acid may be used to dissolve basic metal salts (hydroxides, oxides, and carbonates). Sulfuric acid is the most widely used in industrial leaching operations (it is, however, restricted to metals that do not form insoluble sulfates). Hydrochloric acid is used in conjunction with metal complexing agents. Carbon dioxide may be used with ammonia in the leaching of copper and nickel (DeRenzo, 1978). Acid may also serve to flush some basic organics such as amines, ethers, and anilines (Weininger, 1972).

Sodium hydroxide solutions may also be used to flush certain metals and organics. Sodium hydroxide is used to dissolve metallic aluminum (DeRenzo, 1978) and can be used as well on zinc, tin, lead, and other metals. It may be useful in dissolving certain organic sulfur compounds and phenols (Considine, 1974).

Complexing and chelating agents may also find limited use in a solution mining removal system for heavy metals. Some commonly employed substances are ammonia and ammonium salts, citric acid, ethylene diamine tetracetic acid (EDTA), and thiourea (Rogoshewski and Carstea, 1980). Also, the use of surfactants may prove useful in dissolving oils and greases, such as those found in oily sludges from petroleum refineries.

As the above reagents will be used in the field rather than in the controlled conditions of a chemical processing plant, it is important that safety and pollution control considerations enter into the selection of a particular reagent. For example, hydrochloric or nitric acid may be unsuitable at concentrations where vaporization becomes a problem. It is important to consider the reaction products that could be formed when selecting solvents. For example, the use of hydrochloric acid to flush an amine waste could result in the formation of amine hydrochlorides. It has also been suggested that hydrochloric acid may be used without complexing agents.

An economically feasible process may involve the recycling of elutriate through the contaminated material, with make-up solvent being added to the system while a fraction of the elutriate stream is routed to a portable wastewater treatment system. The appropriate types of wastewater treatment operations will depend on waste stream characteristics, and a discussion of their applications can be found in Appendix B.

The advantages of the process are that, if the waste is amenable to this technique and distribution, collection, and treatment costs are relatively low, solution mining can present an economical alternative to the excavation and treatment of the wastes. It may be particularly applicable if there is a high safety and health hazard associated with excavation. Also, the effectiveness and completion of the treatment process can be measured via sampling prior to wastewater treatment.

Disadvantages include an uncertainty with respect to adequate contact with wastes; that is, because the wastes are buried, it is difficult to determine whether the solvent has contacted all the waste. Also, containerized waste cannot be treated effectively by this method. Another disadvantage is that the solution mining solvent or elutriate may become a pollutant itself if the system has been poorly designed.

Costs for a solution mining clean-up will depend on the amount of waste material to be flushed, the amount of solvent material required, the well-point collection system, the solvent/elutriate distribution system, and the wastewater treatment system. Costs for tile drains and well point dewatering have been previously given in sections on leachate and groundwater control. Unit costs for some potential solvents and reagents are given in Table 7-9. The costs for wastewater treatment can be developed from information given in Appendix B.

TABLE 7-9

COSTS OF POTENTIAL SOLUTION MINING CHEMICALS

Chemical	Unit cost
Hydrochloric acid, 20% acid	$69/ton
Nitric acid, up to 42 Be	$175/ton
Sulfuric acid, virgin	$40 - 65/ton
Sulfuric acid, smelter	$6 - 40/ton
Caustic soda, liquid 50%	$150 - 200/ton
Citric acid	$0.62 - 0.71/lb
Sodium lauryl sulfate, 30%	$0.18 - 22/lb

Source: Schnell, 1980

7.6.2 Neutralization/Detoxification

In-situ neutralization/detoxification is the technique of applying or injecting into the waste disposal site or contaminated area a substance that immobilizes or destroys a pollutant. This technique is restricted to contaminants that can be degraded, have non-toxic breakdown products, and/or are convertible to insoluble precipitates. It has been recommended that this technique be applied to industrial waste disposal sites only, since municipal landfills would constantly be generating anaerobic decomposition products that would require neutralization over a long period of time (Tolman et al., 1978). If the material is present as a solid and is constantly dissolving, a similar problem exists, and neutralization or detoxification would have to be done repeatedly over a period of time. Thus, this method is applicable to what is probably a rare situation, i.e., industrial wastes that are either completely dissolved or readily mobile. A schematic of the process for detoxification of cyanide is shown in Figure 7-15.

As with solution mining, in-situ neutralization or detoxification is highly waste-specific. For example, calcium salt solutions would be used to precipitate free fluorides (Tolman et al., 1978). Many heavy metals may be precipitated as insoluble salts by application of alkalis or sulfides. Sodium hypochloride at 2,500 ppm available chloride has been used to detoxify cyanide contamination by oxidation resulting from indiscriminate dumping (Farb, 1978). Other oxidizing agents that may be used are potassium permanganate or hydrogen peroxide. Hydrogen peroxide has been found to be particularly useful for oxidizing cyanide aldehydes, dialkyl sulfides, dithionate, nitrogen compounds, phenols, and sulfur compounds (FMC Corp., 1979). Reducing agents such as ferrous sulfate may be used in conjunction with hydroxides to insolubilize hexavalent chromium (Tolman et al., 1978; Metcalf and Eddy, Inc., 1972). Heterogeneous mixtures of wastes present a difficult problem because different treatments will be required for different wastes, and treatment for one waste may be unsuitable for another. A further limitation is then introduced, which is that this technique should be used only with well defined, segregated waste cells.

Many of the environmental, health, and safety considerations that apply to solution mining also apply here. In contrast to solution mining, in-situ neutralization/detoxification techniques do not inherently incorporate seepage collection systems. Therefore, if neutralization/detoxification is unsuccessful, any leachate generated will not have been contained. Therefore, an additional fail-safe collection system is required, which is a major disadvantage. Another disadvantage is that it is difficult to determine the degree of effectiveness of this treatment technique. Although the technique is very limited in application, it may prove economically feasible if a known industrial pollutant is dissolved in a disposal site and its location can be well defined.

Economics for a neutralization/detoxification system include the cost for well point injection system, chemical and feed system, and costs for probing, excavation, and drilling. Tolman et al. (1978) has estimated a cost to clean up a hypothetical 10-acre disposal site that had received a single load of cyanide salts in drums that, over time, have dissolved. Costs are summarized in Table 7-10. Costs for other chemicals that may be used in an in-situ neutralization/ detoxification clean-up are shown in Table 7-11.

FIGURE 7-15

IN-SITU DETOXIFICATION PROCESS FOR CYANIDE

(Source: Tolman, et al., 1978)

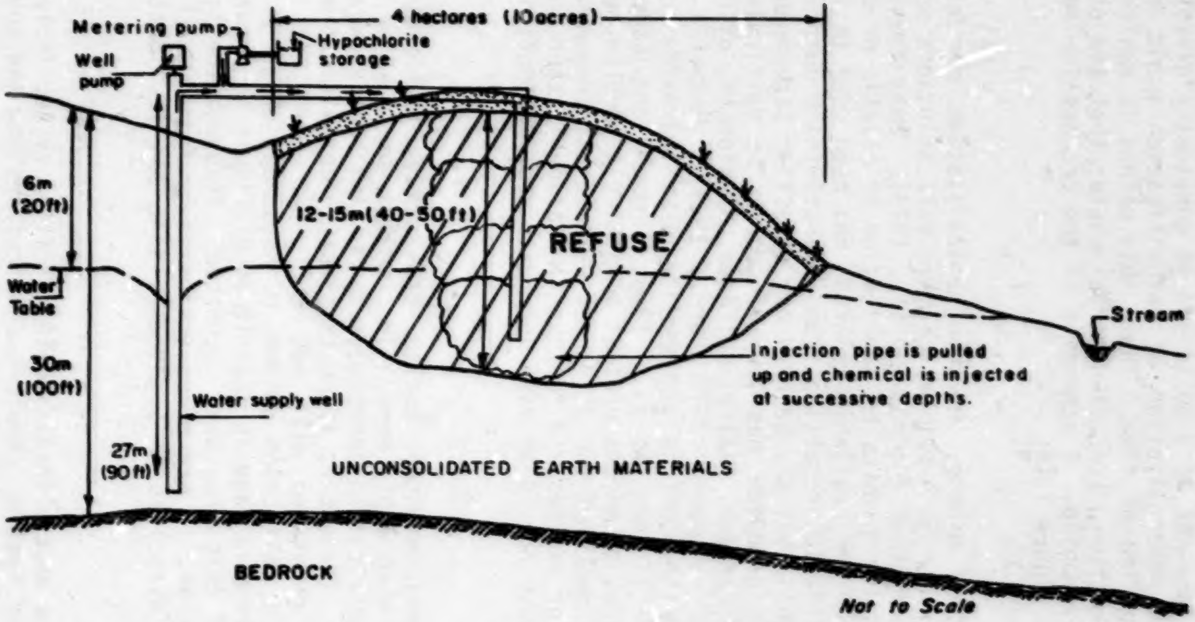

Metering pump
Well pump
Hypochlorite storage
4 hectares (10 acres)
6m (20ft)
Water Table
30m (100ft)
27m (90 ft)
Water supply well
12-15m (40-50 ft)
REFUSE
Stream
Injection pipe is pulled up and chemical is injected at successive depths.
UNCONSOLIDATED EARTH MATERIALS
BEDROCK
Not to Scale

TABLE 7-10

COSTS FOR IN-SITU DETOXIFICATION OF CYANIDE[1]

Item	Cost
Exploratory probing, excavation, and drilling	$15,000
Development of water supply well, 27 m (90 ft); pump and piping	5,000
Installation of 45 well points	10,000
Cost of chemical feed pump	2,000
Cost of chemical (sodium hypochlorite)	5,400
Labor for chemical injection, raising of well points to flood successive elevations (assumed 4 wells handled simultaneously), and general labor (1,600 hours)	48,000
Power (assumed electrical supply available)	500
	$86,300

---

[1] Assumed 10-acre landfill with a total of 1,566 lbs of cyanide distributed within a fill volume of 4.9 million cubic feet. Chemical application rate of 68 gallons per pound of cyanide.

Source: Tolman et al., 1978

TABLE 7-11

COSTS OF POTENTIAL IN-SITU NEUTRALIZATION/ DETOXIFICATION CHEMICALS

Chemical	Unit cost
Calcium chloride, 100-lb bags	$80/ton
Calcium sulfate	$36.20/ton
Potassium permanganate	$1.29/kg
Hydrogen peroxide, 50%	$0.25/lb
Sodium hydroxide, liquid 50%	$150-200/ton
Ferrous sulfate	$80/ton

Source: Schnell, 1980.

7.6.3 Microbial Degradation

Seeding a waste material with microorganisms to achieve degradation may be feasible if the waste has been determined to be biodegradable. Research with the landfarming of oily sludges has shown that many naturally occurring bacteria can be adapted to the breakdown of petroleum constituents in soil. These bacteria include Pseudomonas, Nocardia, Arthrobacter, Flavo bacterium, and Corynebacterium (Kincannon, 1972). Specialized strains of bacteria have been developed for the breakdown of specific chemicals (see Appendix B) and may be ordered in dry bulk quantities (Polybac, 1978). The biodegradation process is slow relative to other remedial actions discussed in this handbook. Complete degradation of the waste could take several years and may never be complete if refractory compounds such as polynuclear aromatics are present (Kincannon, 1972). This is a major disadvantage, since additional migration of contaminants can occur during the treatment and even afterwards. For petroleum sludges, biodegradation is an aerobic process, hence proper aeration is required. This fact probably applies to other organic wastes as well. Therefore, this technique is generally limited to those situations where the waste material or contaminated soil is naturally aerated or where artificial aeration is feasible. Also, the addition of nutrients such as nitrogen and phosphorus may be required if the waste material is deficient in these constituents. Lime will be required to maintain proper pH. Thus, further problems exist, with respect to application and mixing of lime and nutrients. The above criteria suggest that biodegradation may have very limited application as a remedial technique for in-situ direct treatment of waste material. Situations where it could be applied are those where complete mixing and aeration can be achieved, i.e., a chemical spill or landspreading operation where the wastes have not migrated below tilling depth (about 12 inches), or a surface impoundment in which the waste is fluid enough to be mechanically aerated and pumped for mixing. Unit costs for some components of an in-situ microbial degradation system are given in Table 7-12.

TABLE 7-12

UNIT COSTS FOR IN-SITU MICROBIAL DEGRADATION

Component	Unit cost	Reference
Polybac microbes, dry bulk	$23.75/lb	1
Lime	$59/acre	2
Fertilizer	$85/acre	2
Liming (labor & equipment only)	$160/acre	2
Fertilizing (labor & equipment only)	$200-270/acre	2
Seeding (labor & equipment only)	$115-185/acre	2
Hydraulic spreading (labor & equipment only)	$115/acre	2
Soil cultivation (once per month)	$200/acre-month	3
Floating mechanical aerators, stainless, installed		4, 5
10 HP	$6,300 ea.	
50 HP	$11,600 ea.	
100 HP	$21,600 ea.	

---

[1] Polybac Corporation, 1978
[2] McMahon and Pereira, 1979
[3] Kincannon, 1972
[4] Richardson Engineering Services, Inc. 1980
[5] Godfrey, 1979

As an example, assume that an acre of land adjacent to an oily sludge surface impoundment has been contaminated by surface seepage. Contamination has been limited to the top 12 inches of soil due to low soil permeability in the area. It was determined that microbial seeding, liming, and fertilization of the area with subsequent cultivation of the soil could degrade the wastes. If soil was cultivated (aerated) every month and lime and fertilizer were applied every six months for a two-year period, the costs would be as follows:

Initial Application

- Polybac microbes
  - -- assume an application rate of 100 lbs/acre
  - -- 100 lbs x $23.75/lb = $2,375

- Lime and fertilizer materials cost

  -- $59/acre + $85/acre = $144
- Hydraulic spreading = $115
- Soil cultivation = $200

Monthly Soil Cultivation

- 23 months x $200/mo.-acre = $4,600

Semiannual Liming and Fertilization

- Cost of liming = $160/acre + $59/acre = $219/acre
- Cost of fertilization = $270/acre + $85/acre = $355/acre
- Fertilization and liming costs assuming four applications in two years = 4 x ($219 + $355) = $2,296

Total Costs for In-Situ Microbial Degradation (1 acre)

$2,375 + $144 + $115 + $200 + $4,600 + $2,296 = $9,730

7.7 OTHER DIRECT TREATMENT TECHNIQUES

Besides the techniques already mentioned in previous sections, other techniques may be used to control waste from refuse sites. These techniques include:

- Molten Salt
- Plasma Destruction

7.7.1 Molten Salt

The Molten Salt Process is based on the concept of injecting the waste below the surface of a molten salt bath. The salt typically consists of 90 percent sodium carbonate and 10 percent sodium sulfate. The molten salt is placed in a reactor where temperatures are maintained in the range of 1,500°F to 1,800°F. Lower temperatures can be obtained by using a salt with a lower melting point, such as potassium carbonate. Combustion of the wastes may generate gases such as sulfur dioxide or hydrogen chloride. Sulfur dioxide then reacts with the salt to form sodium sulfate, and hydrogen chloride reacts to form sodium chloride.

The molten salt process has the ability to retain particulates generated during the combustion process. Depending on the circumstances, the spent molten salt may be either regenerated or land-disposed.

This process is applicable to disposal of most organic wastes (EPA, 1975). The residence time is usually an average of 3/4 second. The proposed investment cost for a portable unit treating 500 lb/hr is about $500,000 (1975 costs). For a 200 lb/hr unit with salt regeneration, the cost is $800,000 (EPA, 1975).

7.7.2 Plasma Reduction

Plasma is a partially ionized gas composed of ions, electrons, and neutral species. It has been demonstrated that disposal of wastes can be achieved by injecting the waste material in a microwave plasma system (EPA, 1975).

The plasma destruction process is based on the concept that a molecular bond can be broken by transferring the energy of an excited particle to the bond. In a plasma destruction system, microwave energy is applied to excite the molecules of the carrier gas, thus raising the energy levels of the free radicals (EPA, 1975). The excited electron then transfers its energy to break the bonds of materials located in the near proximity. Thus, essentially any organic waste regardless of its physical properties (liquid, solid, gas) may be destroyed by the plasma destruction process.

The residence time within the plasma ranges from 0.1 to 1 second. The operating temperature is low (300°F).

The plasma destruction system is only in the developmental state and has been limited to gaseous material at laboratory-scale operations. This process has a very high potential for becoming a waste disposal process if favorable markets can be developed.

Cost for plasma destruction has been estimated at $.10/lb of waste for electricity and about $10,000 capital cost (EPA, 1975).

REFERENCES

Barnard, W. 1978. Prediction and control of dredged material dispersion around dredging and open-water pipeline disposal operations. U.S. Army Engineer Waterways Experiment Station. Vicksburg, MS.

Brunner D., and D. Keller. 1972. Sanitary landfill design and operation. EPA Report SW-65 ts. Washington, D.C.: U.S. Government Printing Office.

Carson, A. B. 1961. General excavation methods. F. W. Dodge Corporation, N.Y.

Cecos International, Inc., Niagara Falls, New York. February/May 1980. Personal communications with S. Paige and P. Rogoshewski.

Considine, D. (ed.) 1974. Chemical and process technology encyclopedia. New York: McGraw-Hill Book Company.

Day A. 1973. Construction equipment guide. New York: Wiley-Interscience Publications.

DeRenzo D. (ed.) 1978. Unit operations for treatment of hazardous industrial wastes. Noyes Data Corporation, Park Ridge, NJ.

Environmental Concern, Inc. 1980. Scope of services available. St. Michaels, MD.

Farb D. 1978. Upgrading hazardous waste disposal sites: remedial approaches. U.S. Environmental Protection Agency, Cincinnati, Ohio. SW-677.

FMC Corporation. 1979. Industrial waste treatment with hydrogen peroxide. Industrial Chemicals Group. Phildelphia, Pennsylvania.

Godfrey R. (ed.). 1976. Building construction cost data: 1976 cost catalogue. Duxbury, MA: Robert Snow Means Co., Inc.

Godfrey R. (ed.). 1979. Building construction cost data, 1980. Kingston, MA: Robert Snow Means Co., Inc.

Gren G. 1976. Hydraulic dredges, including boosters. In: Proceedings of the Specialty Conference on Dredging and its Environmental Effects. New York: American Society of Civil Engineers.

Huston J., and W. Huston. 1976. Techniques for reducing turbidity associated with present dredging procedures and operations. Prepared for U.S. Army Engineer Waterways Experiment Station. Vicksburg, MS. Contract no. DACW 39-75-0073.

Kincannon C. 1972. Oily waste disposal by soil cultivation process. U.S. Environmental Protection Agency, Washington, D.C. EPA-R2-72-110.

Linsley R., and J. Franzini. 1979. Water-resources engineering, 3d ed. New York: McGraw-Hill Book Company.

Lubowitz H., R. Derham, L. Ryan, and G. Zakrzewski. 1977. Development of a polymeric cementing and encapsulating process of managing hazardous wastes. Prepared under contract 68-03-2037 by TRW Systems Group for USEPA/ORD Municipal Environmental Research Laboratory, Cincinnati, OH.

Lubowitz H., and C. Wiles. 1978. Encapsulation technique for control of Hazardous Wastes. In: Land disposal of hazardous wastes: Proceedings of the Fourth Annual Research Symposium. Shultz, D. et al. (eds.). Cincinnati, Ohio: Municipal Environmental Research Laboratory, ORD. EPA-600-9-78-016.

McMahon, L., and P. Pereira (eds.). 1979. 1980 Dodge guide to public works and heavy construction costs. New York: McGraw-Hill Information Systems.

Metcalf and Eddy, Inc., 1972. Wastewater engineering: collection, treatment, disposal. New York: McGraw-Hill Book Company.

National Car Rental System Inc., Mudcat Division, Fort Lee, New Jersey. January 1980. Personal communication with P. Rogoshewski.

Palesh and Gulledge, 1979.

Pojasek, R.B. April 1978. Stabilization, solidification of hazardous wastes. Environmental Science and Technology 12(4):382-386.

Polybac Corporation. 1978. Technical data sheets. Allentown, PA.

Pradt L. A. 1972. Developments in wet air oxidation. Chemical Engineering Progress 68(12):72-77. [Updated 1976.]

Richardson Engineering Services, Inc. 1980. Process plant construction estimating standards, vol. 1. Solana Beach, CA.

Rogoshewski, P.J., and D.D. Carstea. 1980. An evaluation of lime precipitation as a means of treating boiler tube cleaning wastes. U.S. Environmental Protection Agency, Research Triangle Park, NC. EPA-600-7-80-052.

Sato E. 1976. Application of dredging techniques for environmental problems in dredging: Environmental effects and technology. San Francisco: WODCON Association.

Schnell Publishing Co. 1980. Chemical Marketing Reporter 217(22): [2 June issue].

Stubbs F. W. 1959. Handbook of heavy construction, 1st ed. New York: McGraw-Hill.

Tolman, A., A. Ballestero, W. Beck, and G. Emrich. 1978. Guidance manual for minimizing pollution from waste disposal sites. U.S. Environmental Protection Agency, Cincinnati, Ohio. EPA-600/2-78-142.

Vaughan Co., Inc. 1980. Vaughan pumps. Montesano, WA: Vaughan Co., Inc.

Vaughn-Maitlen Industries (VMI), Bethany, OK. March 1979. Personal communication with P. Rogoshewski.

Weininger, S. 1972. Contemporary organic chemistry. New York: Holt, Rinehart, and Winston.

U.S. Environmental Protection Agency. 1980. Hazardous wastes and consolidated permit regulations. Federal Register 45(93):33063-33578.

U.S. Environmental Protection Agency. 1975. Incineration in hazardous waste management. Office of Solid Waste Management Programs. Municipal Environmental Research Laboratory, Cincinnati, OH. SW141.

U.S. Environmental Protection Agency. 1978. Liners for sanitary landfills and chemical and hazardous waste disposal sites. Cincinnati, OH. PB 293335.

U.S. Environmental Protection Agency. 1979a. Process design manual: municipal sludge treatment and disposal. Municipal Environmental Research Laboratory, Cincinnati, OH. EPA-625/1-79-011.

U.S. Environmental Protection Agency. 1979b. Survey of solidification/stabilization technology for hazardous industrial wastes. Cincinnati, OH. EPA-600/2-79-056.

8.0 CONTAMINATED WATER AND SEWER LINES

Sanitary sewers and municipal water mains located down-gradient from hazardous waste disposal sites may become contaminated by infiltration of leachate or polluted groundwater through cracks, ruptures, or poorly sealed joints in piping. Water mains are less susceptible to the inflow of liquid contaminants, since they are generally full-flowing, pressurized systems constructed of cast iron piping with high structural integrity. They are much more resistant to cracking from root intrusion or settlement than are the vitrified clay pipes (VCP) commonly used for gravity sewer lines. However, the contamination of municipal mains carrying a potable water supply to commercial and residential consumers has much graver public health consequences than does the contamination of sewage flowing to a treatment plant.

When contamination of sewers or water lines occurs, two basic remedial options are available: (1) in-place cleaning and repair, or (2) removal and replacement. Pipeline rerouting and the construction of impermeable subsurface barriers such as grout curtains are expensive preventive measures that may be implemented when existing sewer and water lines are situated in the proximity of uncontrolled hazardous waste disposal sites.

In order to assess the success of the remedial action utilized, the contaminated source must be tested. The overall success is easiest to determine when results of tests can be evaluated against test results obtained prior to the remedial action. The last section (Section 8.5) of this chapter discusses the monitoring of water and sewer lines near waste disposal sites to assess potential contamination and remedial action effectiveness.

8.1 IN-SITU CLEANING

The methods used to clean, inspect, and repair clogged or leaking sewer lines are well established and, to a large degree, can be applied to rehabilitation of contaminated water lines. Available sewer-cleaning techniques include mechanical scouring, hydraulic scouring and flushing, bucket dredging, suction cleaning with pumps or vacuums, chemical absorption, or a combination of these methods. Access to sewer lines for interior cleaning and repair is most commonly afforded by manholes. Flushing inlets and unplugged residential service connections provide additional points of access. Fire hydrant connections allow access to municipal water lines.

8.1.1 Mechanical and Hydraulic Scouring

Mechanical scouring is effective in removing pipeline obstacles such as roots, stones, and corrosion nodules, greases and sludges. In the case of sewer lines infiltrated by contaminated groundwater or leachate, interior scouring may be necessary to loosen or remove solidified masses of toxic chemical precipitates, which are then flushed or dredged from the line. Scouring, and internal cleaning, in general, is necessary in small-diameter pipelines to facilitate the precise locating of inflow leaks by closed-circuit television inspection. Mechanical scouring techniques include the use of power rodding machines ("snakes"), which pull or push scrapers, augers, and brushes through the obstructed line (Figure 8-1). "Pigs" are bullet-shaped plastic balls lined with scouring strips that are hydraulically propelled at high velocity through water mains to scrape the interior pipe surface (Pawtowski, 1980).

Hydraulic scouring of contaminated lines can be achieved by running high-pressure fire hoses into sewer lines through manholes and flushing out given sections of the sewer. This technique is often used after mechanical scouring devices have cleared the line of solid debris or loosened contaminated sediments and sludges coating the inner surface of the pipe.

8.1.2 Bucket Dredging and Suction Cleaning

A bucket machine can be used to dredge grit or contaminated soil from a sewer line (Figure 8-2). Power winches are set up over adjacent manholes with cable connections to both ends of the collection bucket, which is pulled through the sewer until loaded with debris. The same technique can be used to pull "sewer balls" or "porcupine scrapers" through obstructed pipes (Hammer, 1975). Bucket dredging is also useful for collecting samples of contaminated sediments, groundwater, or leachate that may have infiltrated the lines.

Suction devices such as pumps or vacuum trucks also may be used to clean sewer lines of toxic liquids and debris. Again, manholes and fire hydrants provide easy access for the setup and operation of such equipment.

8.1.3 Chemical Treatment

Another method of sewer pipeline cleaning is the use of hydrophilic polymers--foams and gels--that absorb and physically bind liquid pollutants in a solid elastomeric matrix (Johnson, 1980). These polymers are special chemical grouts that can either be applied internally to pipelines or injected through breaks in the line from the exterior. Once the absorbent grout has set (solidified), the solid grout/pollutant matrix can be hydraulically flushed from the line. The applications of many of these hydrophilic grouts,

FIGURE 8-1

POWER RODDING MACHINE

(Source: Hammer, 1975)

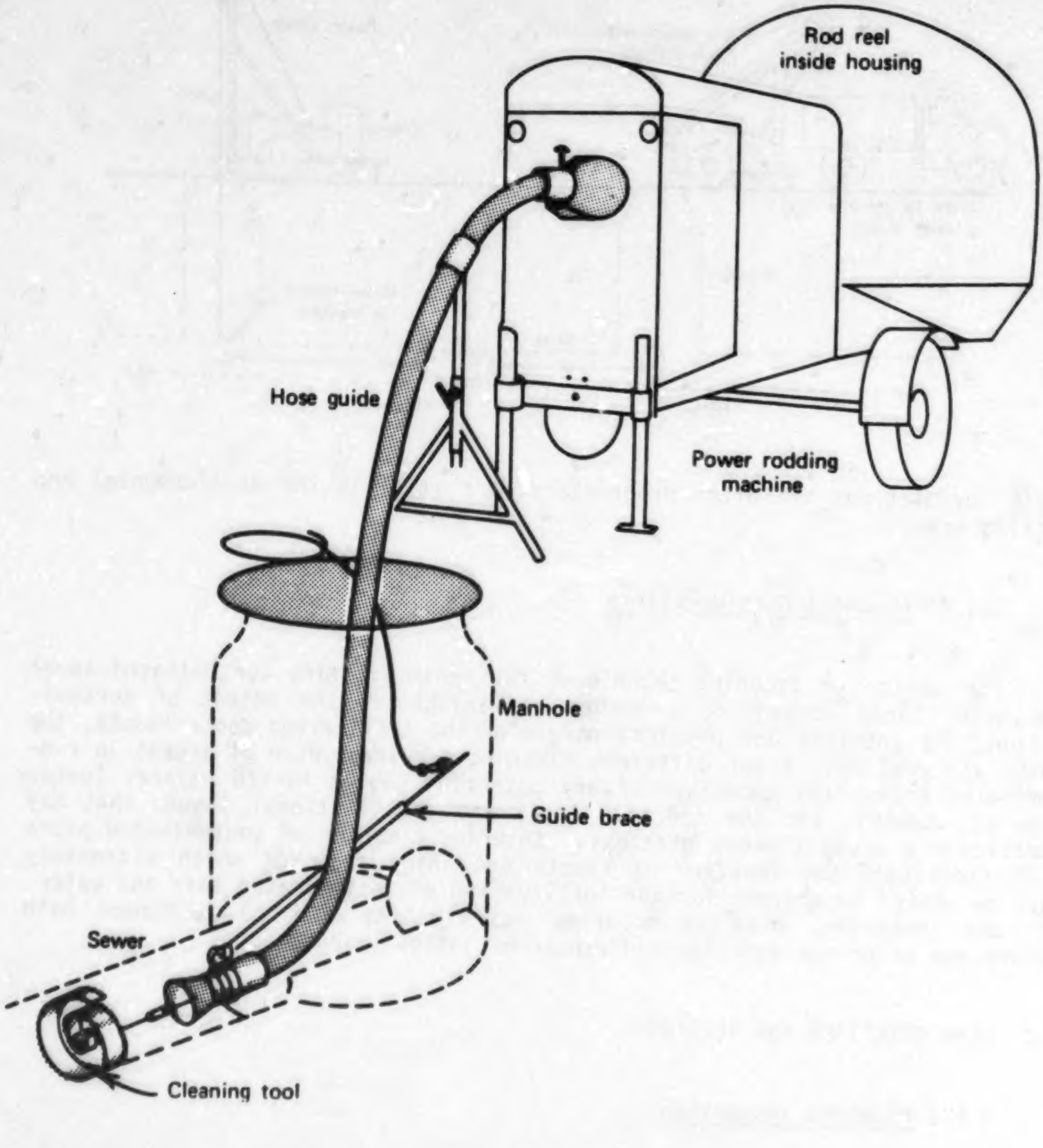
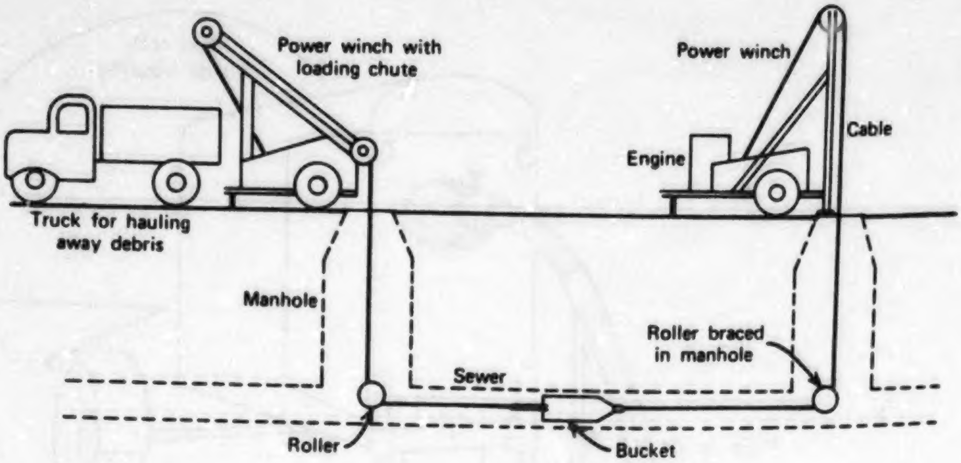

Rod reel
inside housing
Hose guide
Power rodding
machine
Manhole
Guide brace
Sewer
Cleaning tool

FIGURE 8-2

SCHEMATIC OF BUCKET MACHINE CLEANING

(Source: Hammer, 1975)


Power winch with loading chute
Power winch
Cable
Engine
Truck for hauling away debris
Manhole
Roller braced in manhole
Sewer
Roller
Bucket

whose formulations are often proprietary, are still in the developmental and testing stages.

8.1.4 Cleaning Considerations

The choice of cleaning techniques for rehabilitating contaminated sewer and water lines depends on a number of variables: the extent of contamination, the chemical and physical nature of the infiltrated contaminants, the costs and availability of different cleaning services, ease of access to contaminated areas, the immediacy of any potential public health hazards (water line vs. sewer), and the specific legal and constitutional issues that may complicate a given cleanup strategy. Interior cleaning of contaminated pipes will facilitate the location of cracks and joint failures which ultimately must be sealed to prevent further infiltration of contaminated soil and water. In some instances, pipeline scouring and flushing must be performed both before and after the location and repair of inflow leaks.

8.2 LEAK DETECTION AND REPAIRS

8.2.1 Pipeline Inspection

Inspection of pipelines for leaks or infiltration points may be part of a regular sewer or water line maintenance program. Methods to detect and locate pipeline breaches include the use of dyes and other tracer chemicals, patented audiophone leak detectors, smoke testing, and installation of pressure gages along a given length of pipe to monitor changes in hydraulic gradient (Linsley and Franzini, 1979). Large-diameter pipeline interiors are inspected by pulling skid-mounted miniaturized closed-circuit television cameras through the line. The entire inspection can be recorded on videotape for future reference.

Closed-circuit inspection is often performed in conjunction with interior pipeline grouting systems to seal leaks from the inside.

8.2.2 Grouting

Once infiltration points have been specifically located, or cracked or ruptured sections of lines have been discovered, the pipeline can be repaired. One method of in-place repair is to grout fractures or leaky joints to seal and waterproof points of infiltration/exfiltration. Chemical grouts for sealing sewer lines are generally limited to acrylamide resins and silica gels, which are applied to pipeline cracks or non-watertight joints from the interior as they are detected by television inspection. A grouting packer is pulled through the sewer line along with the closed-circuit camera, and as hydraulic breaks or fissures are detected, the grout packer inflates and injects a root killer (such as copper sulfate) and plastic gel or resin into the area, sealing the break (Pawtowski, 1980). Usually, enough chemical grout will set in the surrounding soil to effect a more permanent seal.

Sewer line leaks can be sealed from the exterior by injecting a grout "collar" into the soil immediately surrounding the area of infiltration. The choice of soil grout used for exterior sealing depends largely on the texture, pH, and water content of the surrounding soil. For coarse sands or gravels, mixtures of clay or bentonite and cement are recommended. Finer-grained sands and silty soils are more effectively grouted with dilute silica gels or acrylamide resins (AFTES, 1976). The application of exterior soil grouting to seal leaks generally is limited to gravity sewer lines or storm sewers; full-flowing pressurized sewer lines or water mains are most effectively sealed by interior relining (Hudson, 1980).

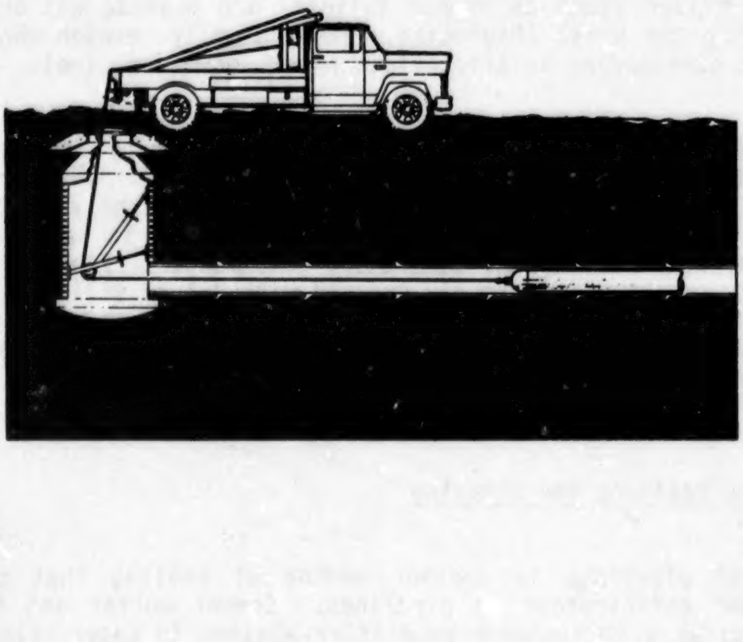
8.2.3 Pipe Relining and Sleeving

Relining of pipelines is another method of sealing that can inhibit infiltration and exfiltration in pipelines. Cement mortar and bitumen are commonly applied as a corrosion-preventative coating in water mains, often by proprietary lining machine processes that do not require removal of the pipe. Interior lining of sewers can be performed in addition to chemical grouting to ensure a high level of pipeline integrity and low future risk of groundwater or leachate infiltration. Large sections of badly cracked or deteriorating sewer lines can be relined with high density polyethylene (PE) piping. Polyethylene is extruded from a special cylinder that is pulled through the lines on a cable (Figure 8-3). As the plastic is extruded, the cylinder is heated and the polyethylene is fused onto the inner surface of the pipe. Once the plastic has cooled and solidified, the cylinder is pulled onto a new section of the line.

Exterior polyethylene sleeving is another method used for leakproofing water mains. Sheets of polyethylene plastic are simply wrapped around the length of pipe sections and taped down. This job is usually performed on new pipeline used for replacement when sections of old or ruptured line are excavated for repair. Sleeving is usually placed only on cast-iron mains 24 inches in diameter or smaller; larger cement mains are not sleeved (Laugle, 1980). The costs of sleeving are minor when compared to the overall costs of pipeline excavation and replacement; sleeving represents a very simple and inexpensive sealing technology.

FIGURE 8-3

INTERIOR RELINING WITH PE



8.3 REMOVAL AND REPLACEMENT

If the inflow of contaminated liquids or sediments is limited to a single section of piping, or if chemical analysis reveals that no public health hazard is posed, and the line is accessible without excavation, then mechanical scouring and/or hydraulic flushing of the interior followed by cement or bitumen relining may be sufficient. However, if the main has ruptured or is known to be an old line, or if extensive contamination of several sections of pipeline has occurred, then excavation and replacement must be performed. Excavation and replacement of contaminated lines is the most expensive remedial option, but in many cases, it is the only available choice to protect public health, especially when water mains have ruptured or have been contaminated by infiltration during pump failures. When water lines break or when there is reason to suspect possible contamination due to the infiltration of polluted groundwater or soil, public works officials must: (1) locate and isolate the affected area(s) of pipeline; (2) shut down pumps and close valves to dry the affected area(s); (3) evaluate the quantity and nature of any infiltrated contaminants (a process that may involve sample collection and analysis), and (4) determine the safest and most economical corrective measure.

Sewer lines that are clogged, contaminated, or cracked generally are much easier to access than water lines for in-place cleaning and repair. Excavation and replacement of buried sewer lines is a much more costly alternative since these are generally laid deeper in the ground than water lines. Also, in-place rehabilitation methods for sewer lines are well-established. For pressurized water mains, however, cracks and ruptures represent a much more serious problem in terms of an interrupted or potentially contaminated public water supply. Excavation may be necessary to access failed joints in water pipelines, and ruptured or seriously contaminated lines must be replaced.

8.4 COSTS

A summary of unit costs associated with pipeline cleaning and repair is presented in Table 8-1.

The costs presented in Table 8-1 represent average unit costs for sewer and water line rehabilitation techniques. Some of the costs are derived from sources in specific geographic localities (i.e., Cincinnati and Washington, D.C.) and may not reflect national averages for the services indicated. Site-specific complications may add to these unit costs. For instance, if a pipeline is situated beneath surface pavement, excavation costs will increase significantly. Costs for removal of surface pavement vary depending on thickness and type of material. Removal of bituminous driveway and road pavement costs approximately \$1.50 to \$2.00 per square yard of material excavated. Excavation of reinforced concrete (up to 6 inches thick) or sidewalk concrete will add \$3.00 to \$4.00 per square yard of removed surface. Excavation through

TABLE 8-1

COSTS FOR CLEANING, REPAIR, AND REPLACEMENT OF WATER AND SEWER LINES

Remedial technique	Cost[1]	Source of cost information
Sewers		
Hydraulic or mechanical scouring, 12" VCP line	$2.00/ft	D.C. area sewer service contractor
Television inspection, 12" line	$1.00/ft	D.C. area sewer service contractor
Chemical grout for sewer line sealing	$10-$15/gal or $3.00/lb	Avanti International acrylamide grout
Clean, T.V. inspect and grout sewer line	$5.00/ft average[2]	D.C. area sewer service contractor
Laying VCP in trench (not including excavation and backfill)	8" pipe: $6/ft 12" pipe: $10/ft 24" pipe: $38/ft 36" pipe: $100/ft	R. Godfrey (ed.), Building Construction Cost Data, 1980
Reinforced concrete pipe (storm sewer), 8' lengths, trench excavation, pipe placement, backfill and compaction	36": $46/ft 42": $57/ft 48": $73/ft 54": $90/ft 60": $119/ft	1980 Dodge Guide
Water mains		
Ductile iron pipeline excavation and replacement (including polyethylene sleeving)	8" pipe: $50/ft 12" pipe: $60/ft 16" pipe: $80/ft 24" concrete pipe: $100/ft	Cincinnati Water Works Bids (EPA/MERL)
In-placing cleaning and cement re-lining of pipes	6-12" pipe: 50% of replacement 24" pipe: 25-30% of replacement	Cincinnati Water Works Bids (EPA/MERL)
Laying ductile iron pipe, 13' lengths (not including excavation and backfill)	8" pipe: $13/ft 12" pipe: $21/ft 16" pipe: $30/ft 24" pipe: $46/ft	R. Godfrey (ed)., Building Construction Cost Data, 1980

--continued--

TABLE 8-1 (Continued)

Remedial technique	Cost	Source of cost information
Trench excavation and backfill		
4 ft. wide trench excavated 4 to 10 ft deep in sandy loam	\$2-\$3/yd[3]	R. Godfrey (ed.), Building Construction Cost Data, 1980
Trench backfill by hand; hand tamp compaction	\$13-\$15/yd[3]	R. Godfrey (ed.), Building Construction Cost Data, 1980

[1] Unit costs include materials and labor for services indicated.
[2] Cost will vary depending on size of line and amount of grouting required; larger pipe, higher costs.

thicker layers of concrete may cost more than \$50 per cubic yard of removed material (Godfrey, 1979). Dewatering of trenches during excavation using diaphragm or centrifugal pumps will increase costs by \$100 to \$300 per day depending on pump size required (Godfrey, 1979).

The costs presented above also do not include those for hauling of materials (e.g., pipeline) to the job site, or the necessary handling and disposal of possibly hazardous debris that may be scoured and flushed from contaminated lines. Where soil grouting is to be performed to collar a sewer line break, the choice of an appropriate grout may require a geotechnical evaluation of the soils in question, thus presenting another consideration that may add significantly to total costs.

8.5 MONITORING

A properly planned monitoring program is essential in the determination of the extent and magnitude of water and sewer line contamination. Collected data can also be used as an early warning system of contamination as well as an evaluation tool in the determination of the effectiveness of remedial actions.

The planning of a monitoring program should address three main criteria: (1) sampling location, (2) parameters to be measured and analytical considerations and (3) sampling frequency. Those criteria are discussed in more detail in the following sections.

8.5.1 Sampling Locations

Sampling locations should be strategically located so that the results of sampling give an accurate description of the extent of contamination as well as identify the most probable contamination source (i.e., where the contaminants are entering the water or sewer lines). Because of the differing nature of water and sewer lines (usually pressurized versus gravity flow), the access for sampling locations will also differ.

Sampling of water lines may be most easily accomplished by obtaining samples from private residences and from commercial and industrial establishments. In most instances, permission to sample these sources can be easily obtained, especially since there is usually no cost involved for the party being sampled and since the party generally knows that the program is being carried out for his benefit. Where municipal sources, such as fire hydrants are available, these can be utilized also.

In obtaining samples from the sources previously mentioned, it is very important to establish whether the water has been treated. At private residences treatment may include removal of iron , neutralization, water softening, and/or water filtration. Water treatment at industrial locations may be more complex, depending on the use of the water in the industrial process. The sample must represent untreated water to insure that none of the contaminants has been removed.

In choosing the sampling points, the location of the disposal site in relationship to groundwater gradients must be assessed. The most effective program would test the nearest down-gradient water and sewer lines. These are the ones that would most likely be contaminated first and would provide the earliest warning. This, of course, is true assuming that these lines are representative of the condition of other lines nearby. In other words, a new water or sewer line near a contaminating source may not be contaminated, but a line in poor condition but further from the source may become contaminated.

Sampling of sewer lines can be accomplished at access points such as manholes and at the treatment plant. Treatment plant monitoring is carried out at most plants on a routine basis. Often it is during the analysis of these samples that contamination is detected. The results from a treatment plant do not, however, identify the location of the contaminating source. Therefore, once contamination has been identified, the system has to be reviewed to identify the areas most likely to be contaminated. Additional samples should then be collected from manholes located in the area to pin-point the contaminating source.

8.5.2 Parameters to be Measured and Analytical Considerations

The selection of particular parameters to be measured depends upon the nature of the suspected source. The selection of parameters should assure that the contamination can be easily detected, and should identify some of the more toxic species so that potential health effects can be mitigated.

Substances of potential concern in hazardous waste leachate include:

- Soluble, oxygen-demanding organics
- Soluble substances that cause tastes and odors in water supplies
- Color and turbidity
- Nutrients such as nitrogen, phosphorus, and carbon
- Toxic organic and inorganic substances
- Refractory materials
- Oil, grease, and immiscible liquids
- Acids and alkalis
- Substances resulting in atmospheric odors
- Suspended and dissolved solids

The final list of parameters chosen for analysis should consider the following (EPA, 1980):

- Electrical conductivity (water and sewer lines)
- Turbidity (water lines)
- Settleable, suspended, and total dissolved solids (waterlines)
- Volatile solids (water lines)
- Oils, greases and immiscible liquids (water and sewer lines)
- Odors (water and sewer lines)
- pH (water and sewer lines)

- Oxidation Reduction Potential (ORP) (water and sewer lines)
- Acidity (water and sewer lines)
- Alkalinity (water and sewer lines)
- Biochemical Oxygen Demand (BOD) (water and sewer lines)
- Chemical Oxygen Demand (COD) (water and sewer lines)
- Total Organic Carbon (TOC) (water and sewer lines)
- Heavy metals (water and sewer lines)
- Other specific inorganic sustances (water and sewer lines)
- Nitrogen and phosphorus compounds (water and sewer lines)
- Volatile organic acids (water and sewer lines)
- Toxicity (water and sewer lines)

Analytical procedures have been suggested by EPA and ASTM. The proper techniques (acceptable to the regulatory agencies) must be utilized. Among the factors to be considered in selection of an analytical method are:

- Sensitivity, precision and accuracy required
- Interferences
- Number of samples to be analyzed
- Quantity of sample available
- Analytical turn-around time
- Analytical cost

8.5.3 Sampling Frequency

Sampling frequency is largely dependent on the type of monitoring program being instituted (e.g., routine detective monitoring, problem identification monitoring, remedial action effectiveness evaluation monitoring). Within a single monitoring program there may be a variety of sampling frequencies for individual sampling locations. The type of sample collected (i.e., grab sample versus a composite sample) must also be established and should suit the objectives of the program.

During a routine monitoring program, to detect potential contamination, samples are often collected monthly (or less frequently). These samples are usually grab samples. If contamination is noted, then the sampling frequency and number of sampling locations should be increased to identify the contaminating source. If water lines are contaminated, an alternative water source may have to be instituted until the problem is corrected. Sampling should be carried out after completion of the remedial action to ensure that the actions taken were effective and sufficient.

Composite samples should be considered in sewer lines where the composition and concentrations display great fluctuation. Along with the composite samples, grab samples should also be collected so that peaks and maximum concentrations of contaminating parameters may be detected.

REFERENCES

AFTES (Association Francaise des Travaux en Souterrain). 1975. Recommendations for the use of grouting in underground construction, trans. G.W. Clough

Godfrey, B. (ed.). 1979. Building Construction Cost Data, 1980. Kingston, MA: Robert Snow Means Co., Inc.

Hammer, M. 1975. Water and waste-water technology. New York: John Wiley and Sons.

Hudson, L., Department of Public Works, City of Baltimore, MD. March 1980. Personal communication with H. Bryson.

Johnson, J., Chemical Research Division, 3M Company. March 1980. Personal communication with H. Bryson.

Laugle, M. for R. Clark. EPA Municipal Environmental Research Laboratory, Cincinnati, OH. February 1980. Personal communication to R. Wetzal.

Linsley, R., and J. Franzini. 1979. Water resources engineering, 3d ed. New York: McGraw-Hill Book Co.

National Association of Sewer Service Companies (NASSCo). 1978-1979. Notes from Underground. Washington, DC. [1978 and 1979 Newsletters.]

Pawtowski, C., J-P Servorooter, Inc. Rockville, MD. March 1980. Personal communication with H. Bryson.

U.S. Environmental Protection Agency. 1980. Management of hazardous waste leachate. R.J. Shuckrow et al., for Municipal Environmental Research Laboratory, Cincinnati, OH. SW-871.

9.0 CONTAMINATED SEDIMENTS

Uncontrolled waste disposal sites may directly or indirectly contaminate bottom sediments deposited in streams, creeks, rivers, ponds, lakes, estuaries, and other bodies of water. Sediment contamination by waste disposal sites may occur along several different pathways. Contaminated soil may be eroded from the surface of hazardous waste disposal sites by natural runoff and subsequently deposited in nearby watercourses or sediment basins constructed downslope of the site. Also, existing sediments along stream and river bottoms may adsorb chemical pollutants that have been washed into the watercourse from disposal areas within the drainage basin. Similarly, contaminated groundwater may drain to surface watercourses and the transported pollutants may settle into, or chemically bind with, bottom sediments. Another possible source of sediment contamination is direct leakage or spills of hazardous liquids from damaged or mishandled waste containers; spilled chemicals that are heavier and denser than water will sink to the bottom of natural waters, coating and mixing with sediments. The precise mechanisms by which sediments may become contaminated by disposal site pollutants will depend on site-specific hydrologic variables and the physical characteristics of the polluting chemical(s).

Contaminated sediments, depending on their quantity and nature, may severely disrupt aquatic ecosystems and may even affect public drinking water supplies. Chemicals that settle into the bottom sediments of natural waters may damage or kill benthic organisms, disrupting the aquatic food chain. Fish kills may follow, or, if a toxic chemical has become concentrated through passage along successive trophic levels, people feeding on the contaminated fish or shellfish may be poisoned. More seriously, if contaminated sediments are deposited in water supply reservoirs, a direct threat to the consuming public may result.

Remedial techniques for contaminated sediments generally involve removal and subsequent disposal or treatment of the sediments. Sediment removal methods include well-established excavation and dredging techniques. Dredged materials ("spoil") management includes techniques for drying, physical processing, chemical treatment, and disposal. Treated sediments, or those that have not been severely contaminated, may be used as construction fill and in reclamation projects. Plans to remove and treat contaminated sediments must be designed and implemented on a site-specific basis. Dredging in wetlands may require revegetation of the area.

A knowledge of the physical properties and distribution of contaminated sediments is highly desirable, if not essential, in selecting a dredging technique and in planning the dredging operation. Information on grain size, bed thickness, and source and rate of sediment deposition is particularly useful in this context. Such information can be obtained through a program of bottom sampling or core sampling of the affected sediment.

9.1 MECHANICAL DREDGING

9.1.1 Description and Applications

Mechanical dredging of contaminated sediments should be considered under conditions of low, shallow flow. Dredging should be used in conjunction with stream diversion techniques to hydraulically isolate the area of sediment removal. Under any other conditions, mechanical excavation with draglines, clamshells, or backhoes may create excessive turbidity and cause uncontrolled transport of contaminated sediments further downstream. Stream diversions with temporary cofferdams, followed by dewatering and mechanical excavation of the contaminated sediments, are typical elements of a mechanical dredging operation for streams, creeks, or small rivers. Mechanical excavation can also be used to remove contaminated sediments that have been eroded from disposal sites during major storms and deposited in floodplains or along river banks above the level of base flow.

For streams and rivers that are relatively shallow and whose flow velocity is relatively low, backhoes, draglines or clamshells can be used to excavate areas of the stream bed where sediments are contaminated. The excavated sediments can be loaded directly onto haul vehicles for transport to predesignated disposal areas; however, the excavated material must be sufficiently drained and dried before transport. This consideration represents a major obstacle in such operations. Drained water may contain contaminated sediments in suspension and must be handled accordingly. If the sediments are relatively "young" and unconsolidated, they may be of a consistency that is hard to handle with draglines or clamshells. Additionally, backhoe and dragline operation requires a stable base from which to work. For these reasons, direct mechanical dredging of contaminated sediments in streams is not recommended except for small streams with stable banks, slow and shallow flow, underwater structures, and where the contaminated sediments are relatively consolidated and easily drained. Direct mechanical excavation is also feasible for contaminated sediments deposited on dry river banks or in floodplains.

A more efficient mechanical dredging operation with broader application involves stream or river diversion with cofferdams, followed by dewatering and excavation of contaminated sediments. Such an operation may prove quite costly; however, there is little chance of stirring up sediments and creating downstream contamination. Efficiency of sediment removal is much greater by this method than by instream mechanical dredging without diversion of flow.

Sheet-pile cofferdams may be installed in pairs across streams to temporarily isolate areas of contaminated sediment deposition and allow access for dewatering and excavation (Figure 9-1). Alternatively, a single curved or rectangular cofferdam may be constructed to isolate an area along one bank of the stream or river (Figure 9-2); this method only partially restricts natural flow and does not necessitate construction of a temporary diversion (by-pass) channel to convey entire flow around the area of excavation, as the first method does.

9.1.2 Design And Construction Considerations

During direct mechanical dredging of stream or river sediments, agitation of the bed deposits during excavation may generate a floating scum of contaminated debris on the water surface, particularly if the chemical contaminant is oily or greasy in nature. The installation of a silt curtain downstream of the dredging site will function to trap any contaminated debris so generated; the debris can then be collected through skimming and hauled to special disposal areas.

Similarly, silt curtains can be employed to minimize downstream transport of contaminated sediments. A schematic of a silt curtain is shown in Figure 9-3. It is constructed of nylon-reinforced polyvinyl chloride and manufactured in 90-foot sections that can be joined together in the field to provide the specified length. Silt curtains are usually employed in U-shaped or circular configurations, as shown in Figure 9-4. Silt curtains are not recommended for flow velocities greater than 1.5 feet per second (Barnard, 1978).

If in-stream mechanical dredging is determined to be technically feasible and cost-effective, it can be performed most effectively during periods of low flow--that is, during the driest months of the year for the particular climate. This generalization will hold true for any mechanical dredging operation, including those that involve cofferdam construction and stream diversion.

Sheet-pile cofferdams are generally constructed of black steel sheeting, in thickness from 5- to 12-gage and in lengths from 4 to 40 feet. For additional corrosion protection, galvanized or aluminized coatings are available (ARMCO, 1979). Cofferdams may be either single-walled or cellular, earth-filled in sections. Single-wall cofferdams may be strengthened by an earth fill on both sides. Cellular cofferdams consist of circular sheet-pile cells filled with earth, generally a mixture of sand and clay (Linsley and Franzini, 1979).

FIGURE 9-1

STREAMFLOW DIVERSION FOR SEDIMENT EXCAVATION USING

TWO COFFERDAMS AND DIVERSION CHANNEL

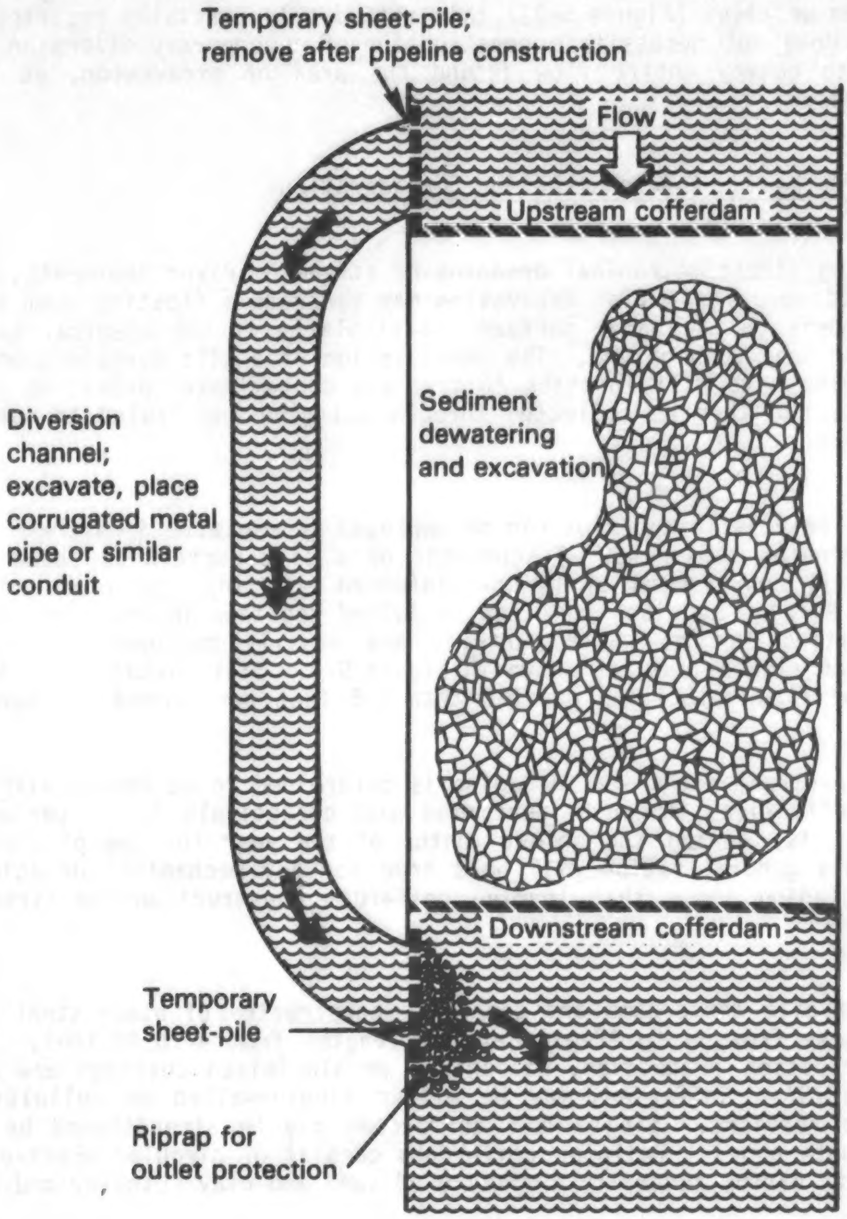

Temporary sheet-pile; remove after pipeline construction
Flow
Upstream cofferdam
Diversion channel; excavate, place corrugated metal pipe or similar conduit
Sediment dewatering and excavation
Downstream cofferdam
Temporary sheet-pile
Riprap for outlet protection

FIGURE 9-2

STREAMFLOW DIVERSION FOR SEDIMENT EXCAVATION USING SINGLE COFFERDAM

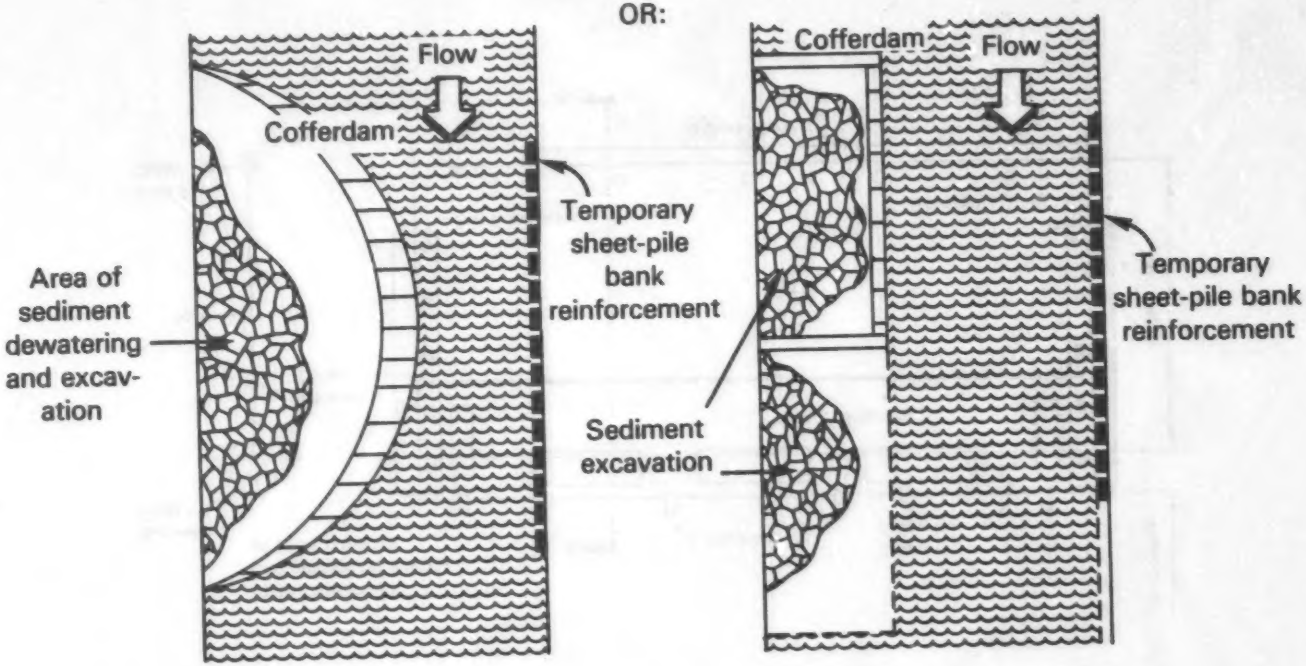

Area of sediment dewatering and excavation
Cofferdam
Flow
Temporary sheet-pile bank reinforcement
OR:
Cofferdam
Flow
Sediment excavation
Temporary sheet-pile bank reinforcement

FIGURE 9-3

CONSTRUCTION OF A TYPICAL CENTER TENSION SILT CURTAIN SECTION

(Source: Barnard, 1978)

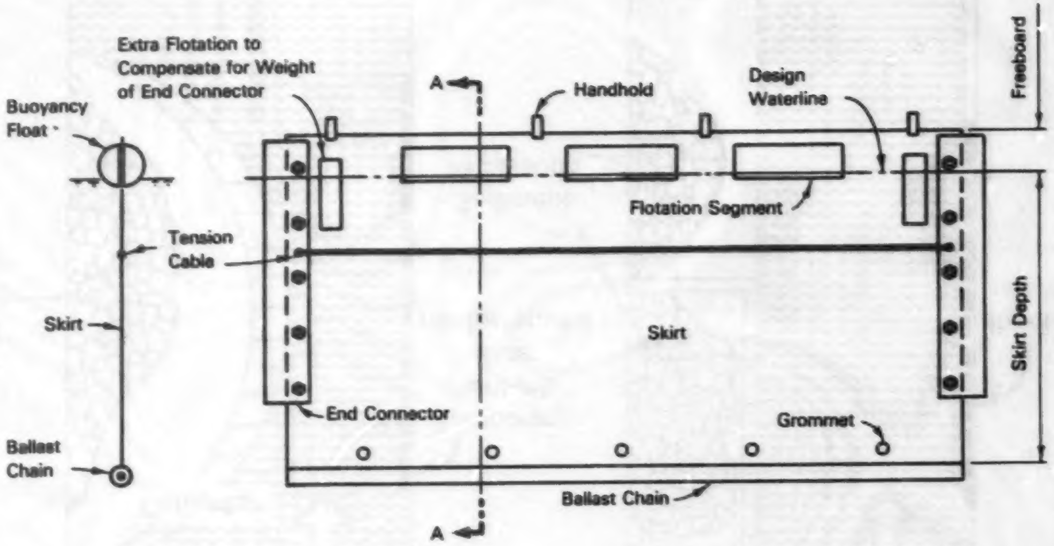
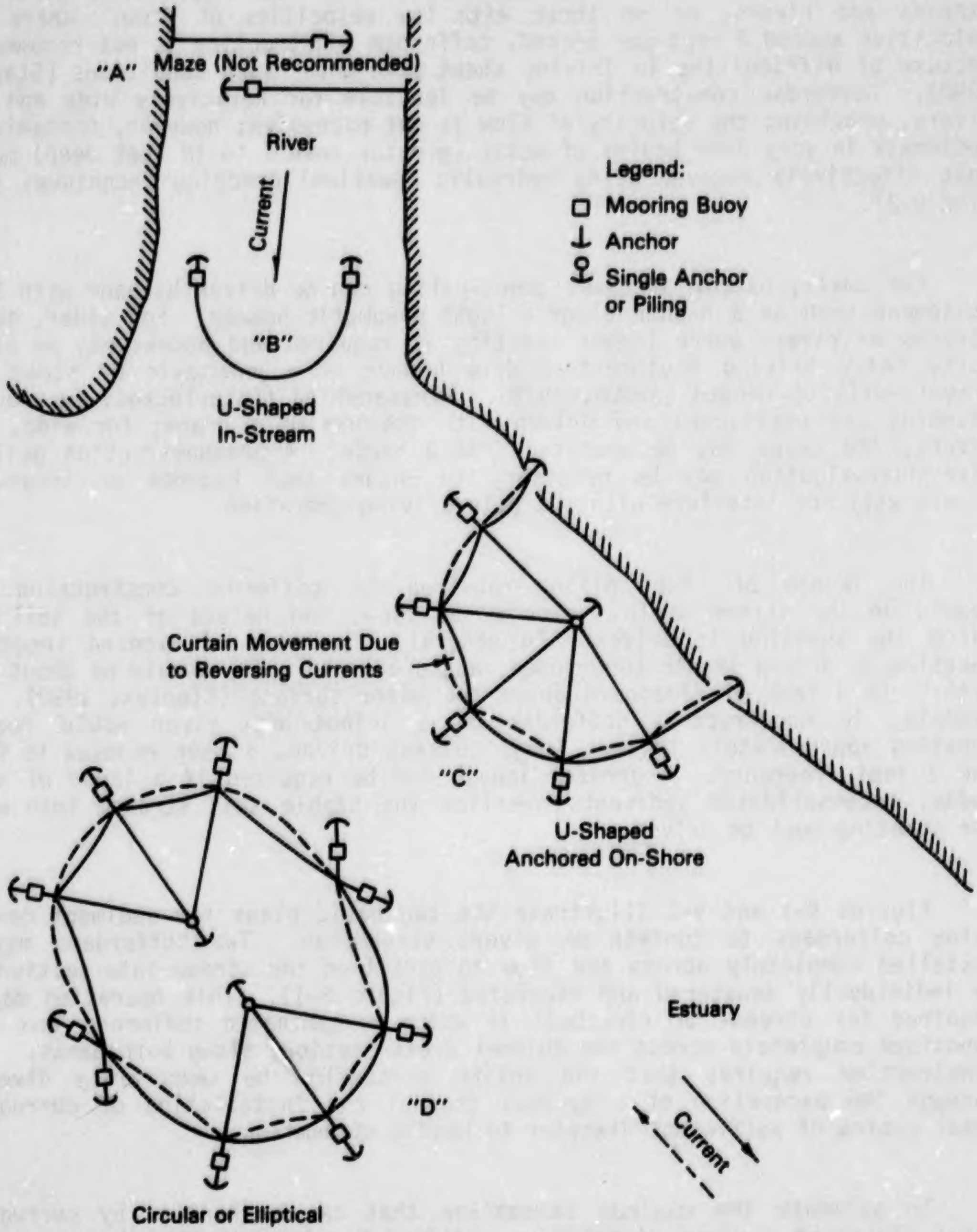

Extra Flotation to Compensate for Weight of End Connector
Buoyancy Float
A
Handhold
Design Waterline
Freeboard
Flotation Segment
Tension Cable
Skirt
Skirt
Skirt Depth
End Connector
Grommet
Ballast Chain
Ballast Chain
A

FIGURE 9-4

TYPICAL SILT CURTAIN DEPLOYMENT CONFIGURATIONS

(Source: Barnard, 1978)


"A"
Maze (Not Recommended)
River
Current
"B"
U-Shaped
In-Stream
Legend:
□ Mooring Buoy
⊥ Anchor
Single Anchor
or Piling
Curtain Movement Due
to Reversing Currents
"C"
U-Shaped
Anchored On-Shore
Estuary
Current
"D"
Circular or Elliptical

Single-wall sheet-pile cofferdams are most applicable for shallow water flows. For depths greater than 5 feet, cellular cofferdams are recommended (Linsley and Franzini, 1979).

Cofferdams are most easily constructed for flow containment on shallow streams and rivers, or on those with low velocities of flow. Where flow velocities exceed 2 feet per second, cofferdam construction is not recommended because of difficulties in driving sheet pile under such conditions (Staples, 1980). Cofferdam construction may be feasible for relatively wide and deep rivers, providing the velocity of flow is not excessive; however, contaminated sediments in very deep bodies of water (greater than 5 to 10 feet deep) may be most effectively removed using hydraulic (suction) dredging techniques (Section 9.2).

For small, narrow streams, sheet-piling can be driven by hand with light equipment such as a hand maul or a light pneumatic hammer. For wider, deeper streams or rivers where longer sheeting is required and access may be difficult, heavy driving equipment--a drop hammer or a pneumatic or steam pile driver--will be needed (ARMCO, 1979). Preassembled (interlocked) sections of sheeting are positioned and driven with the use of a crane; for wide, deep rivers, the crane may be operated from a barge. A preconstruction geologic site investigation may be necessary to ensure that bedrock or impervious strata will not interfere with the pile-driving operation.

The length of sheet piling required for cofferdam construction will depend on the stream depth, velocity of flow, and nature of the soil into which the sheeting is driven. In general, the ratio of exposed length of sheeting to driven length (unexposed, anchored into soil) should be about 1:1, with 1 to 3 feet of freeboard above the water surface (Staples, 1980). For example, to construct a cofferdam on a 5-foot-deep river would require sheeting approximately 12 feet long: 5 feet driven, 5 feet exposed to flow, and 2 feet freeboard. A greater length may be required if a layer of soft, muddy, unconsolidated sediments overlies the stable soil stratum into which the sheeting must be driven.

Figures 9-1 and 9-2 illustrate the two basic plans for sediment removal using cofferdams to contain or divert streamflow. Two cofferdams may be installed completely across the flow to partition the stream into sections to be individually dewatered and excavated (Figure 9-1). This operation may be required for stream- or riverbeds in which contaminated sediments have been deposited completely across the channel cross-section, along both banks. Such construction requires that the entire streamflow be temporarily diverted through the excavation of a by-pass channel and installation of corrugated metal piping of sufficient diameter to handle streamflow.

To estimate the maximum streamflow that can be handled by corrugated metal piping of a given diameter, a modified Manning equation for corrugated pipe flowing half-full at a maximum grade of 10 percent is given below:

$$V = 67.7 \frac{\pi d}{8 + 4\pi}^{2/3} (S)^{1/2}$$

where V = average velocity of flow in feet/second
d = diameter of the pipe in feet
S = slope of the pipeline channel grade (expressed in decimal form; maximum S for which this equation holds true = 0.10)

This modified Manning equation is based on a roughness coefficient (n) of 0.022 for corrugated metal pipe (Linsley and Franzini, 1979). Corrugated metal piping is generally available in sizes up to 6 feet in diameter. Based on pipe diameter and channel grade, velocity (V) of flow through the pipe can be calculated using this equation. Multiplying the velocity (V) in feet/second by the cross-sectional flow area ($\pi d^2/8$ for half-full flow) in square feet will yield the maximum streamflow (in cubic feet/second) that can be handled by the diversion pipeline. For pipeline discharge velocities exceeding 2.0 feet/ second, it may be necessary to stabilize the pipe outlet area with stone riprap to prevent excessive scouring of the natural stream bed.

For excavation of contaminated sediments deposited along only one side of the channel, a single curved or jointed cofferdam can be installed to isolate the construction area from streamflow (Figure 9-2). Such an installation will partially restrict natural flow, creating an increased water level and higher velocity flow within the restricted area of the channel. To prevent bank overflow and excessive erosion resulting from this restricted flow, it is recommended that a sheet-pile containment wall be driven along the channel bank in the area of restricted flow. Both the cofferdam and sheet-pile reinforcement wall can be pulled when sediment excavation has been completed and re-installed further downstream if additional sediment removal is required.

Areas enclosed by cofferdams may require dewatering if infiltration leaks occur through poorly joined sections of sheet piling or if excessive precipitation occurs during excavation activities. Dewatering can be accomplished with single-stage centrifugal pumps, which are available in sizes that can pump up to 5,000 gallons per minute (Richardson Engineering Services, 1980). Natural drainage and evaporative drying may be sufficient to dewater small areas of sediment deposition, but this may require too much time. Streambed sediments isolated by cofferdams must be sufficiently dewatered before excavation of the contaminated sediments can be performed efficiently.

Mechanical excavation of dewatered, contaminated sediments can be accomplished with backhoes, draglines, or clamshells. Crawler-mounted hydraulic backhoes can handle (excavate and load) as much as 200-300 cubic yards of sediment per hour, with bucket capacities as great as 3 1/2 cubic yards.

Draglines can handle up to 400 cubic yards per hour, with typical bucket capacities of 2 to 5 cubic yards (McMahon and Pereira, 1979). Clamshells are generally equipped with buckets from 3/8- to 2-cubic yards in size, and can handle as much as 50 to 100 cubic yards per hour (Godfrey, 1979). Specific limitations of each equipment type have been previously discussed in Section 7.1. Mechanical dredging output rates will vary depending on the size and mobility of the equipment, and on site-specific conditions such as available working area.

Excavated sediments can be loaded directly into haul trucks on-site for transport to special disposal areas. Haul truck loading beds should be bottom-sealed and covered with a tarpaulin or similar flexible cover to ensure that no sediments are lost during transport. The entire loading operation should be performed carefully to avoid uncontrolled spills of contaminated sediments. Windy weather should be avoided during loading to prevent off-site transport of finer sediments and dust. Sediments should not have been dewatered to the point where they are excessively dry or loose.

9.1.3 Advantages and Disadvantages

The advantages and disadvantages associated with mechanical dredging techniques are summarized in Tables 9-1 and 9-2. Table 9-1 addresses direct instream mechanical dredging ("wet excavation"). Cofferdam diversion streamflow, with subsequent dewatering and mechanical excavation of contaminated sediments, is addressed in Table 9-2.

Backhoe Excavation

- 10 yd^3 of sediments x \$1.50/yd^3 to truck-load with backhoe = \$15
- Backhoe mobilization and demobilization = approx. \$150
- Contaminated sediment hauling, 200 miles to secure landfill; bulk density 1 ton/yd^3; 200 miles x \$4/loaded mile = \$800
- Disposal costs; \$240/ton x 1 ton/yd^3 x 10 yd^3 = \$2400

Additional costs to cover removal of the pipe after use and backfilling of the trench will be incurred in most cases.

TABLE 9-1

SUMMARY OF EVALUATION OF INSTREAM MECHANICAL DREDGING OF CONTAMINATED SEDIMENTS

Advantages	Disadvantages
● May be cost-effective for slow, shallow streams or sediments in dry streambeds or floodplains	● Generates excessive turbidity; may cause downstream transport of sediments
● Also effective for small, isolated pools or ponds containing contamminated sediments	● Only feasible for low, shallow flows with stable streambanks and consolidated sediments
● Barge-mounted operations may be used for larger rivers	● May require special dewatering methods (clamshell lift and drain over haul trucks)
	● Efficiency of removal generally poor
	● Generally not recommended for handling contaminated sediments instream

---

TABLE 9-2

SUMMARY EVALUATION OF COFFERDAM METHOD FOR SEDIMENT REMOVAL

Advantages	Disadvantages
● High efficiency of removal; low turbidity	● May be quite costly for deep, wide flows and sites requiring diversion pipeline
● Involves well-established construction techniques	● Not feasible for fast stream flows (>2 ft/sec)
● Structures easily removed and transported	● Not recommended for flows deeper than 10 feet
● Cost-effective for slow-flowing streams and rivers with favorable access (stable banks; open areas)	● Sediment dewatering may be required

--continued--

TABLE 9-2 (Continued)

Advantages	Disadvantages
	● Access for mechanical excavation equipment may be difficult
	● May require large excavation and loading area
	● Transportation costs may be excessive (remote areas)
	● Geologic substrate may prevent sheet pile drive

TABLE 9-3

COSTS FOR MECHANICAL DREDGING/EXCAVATION OF CONTAMINATED SEDIMENTS

Description	Unit cost	Source
Dredging, mobilization and demobilization; add to total	\$5,000-20,000 total	1
Barge-mounted dragline or clamshell, hopper dumped, pumped 1000' to shore dump	\$4.50-6.50/yd^3	1
Bulk excavation; stockpiled or truck loaded:		
hydraulic backhoe, 1-3½ yd^3 bucket	\$1.16-1.78/yd^3	1
clamshell, 1/2 and 1 yd^3	\$2.32-3.47/yd^3	1
dragline, 3/4 and 1½ yd^3	\$1.39-2.06/yd^3	1
For wet excavation;		
clamshell or dragline	add 100%	1
backhoe	add 50%	1
clamshell excavation in sheeting or cofferdam	\$3.50-13.00/yd^3	1

--continued--

TABLE 9-3 (Continued)

Description	Unit cost	Source
Mobilization and demobilization of backhoe or dragline, 1½ yd	\$150 each	1
Earth hauling, 12 yd^3 dump truck; 4-mile roundtrip, 1.6 loads/hr	\$2.34/yd^3	1
Sheet piling, steel, high strength (55,000 psi); temporary installation (pull and salvage):		
20' deep	\$8.24/ft^2	1
25' deep	\$6.63/ft^2	1
	(see also Table 4.5)	
Pile driver; mobilize and demobilize:		
50-mile radius	\$5,700 total	2
100-mile radius	\$9,450 total	2
Corrugated metal pipe, galvanized or aluminized; pipe placement, not including excavation and backfill above top of pipe		
12-inch diam., 16 ga.	\$ 6.70/linear ft.	1
24-inch diam., 14 ga.	\$12.80/linear ft.	1
36-inch diam., 12 ga.	\$28.00/linear ft.	1
48-inch diam., 12 ga.	\$39.00/linear ft.	1
60-inch diam., 10 ga.	\$60.00/linear ft.	1
Stone riprap; dumped from trucks for outlet protection	\$16.65/yd^3	1
Portable centrifugal water pumps (self-priming):		
3-inch; 20,000 gph, 8.5 hp	\$ 78/week	3
4-inch; 40,000 gph, 31 hp	\$225/week	3
6-inch; 90,000 gph, 64 hp	\$245/week	3
8-inch discharge	\$280/week	2
10-inch discharge	\$380/week	2
12-inch discharge	\$500/week	2

--continued--

TABLE 9-3 (Continued)

Description	Unit cost	Source
Pump hose:		
3-inch, 20-ft section	$20.50/week	3
4-inch, 20-ft section	$28.50/week	3
6-inch, 20-ft section	$64.25/week	3
Silt curtain:	$0.50-2.80/ft^2	4
Hauling of contaminated sediments:	$ 2-4/loaded mile	5
	$ 2-4/loaded mile	
Disposal of contaminated sediments:	$240/ton	5

[1] Godfrey, 1979
[2] McMahon and Pereira, 1979
[3] Richardson Engineering, 1980
[4] Kepner Plastic Fabrications, 1980
[5] Cecos International, Inc. 1980

9.1.4 Costs

Unit costs associated with mechanical dredging techniques are presented in Table 9-3. An example cost calculation using those unit costs for mechanical dredging at a hypothetical site is given below.

* * *

Assume that a stream 7.5 feet wide and 2 feet deep with an average flow velocity of 1 fps (foot per second) contains 10 cubic yards of contaminated sediments deposited along a 30-foot length of the stream from bank to bank. Two sheet-pile cofferdams are to be constructed across the stream to partition the area of deposition, necessitating temporary streamflow diversion with a corrugated metal pipeline. Stream banks are sparsely vegetated and stable, providing easy access for a 2-cubic yard hydraulic backhoe, which will excavate and truck-load the sediments for offsite transport to a secure chemical disposal site 200 miles away. Costs for this operation are derived as follows:

Cofferdam installation and streamflow diversion

- 5' deep x 8' wide = 40 ft^2 x 2 cofferdams = 80 ft^2;
  pile drive and pull after operation is complete;
  80 ft^2 x \$8.24/ft^2 = \$659

- Excavation for pipeline (no backfill);
  assumed trench cross section of 6 ft ;
  30 ft x 6 ft^2 x $\frac{yd^3}{27}$ ft^3 x \$1.78/yd^3 = \$12

- Corrugated metal pipeline, 30' long;
  12-inch diameter will handle streamflow;
  30 ft x \$6.70/ft = \$201 to lay pipeline

9.2 LOW-TURBIDITY HYDRAULIC DREDGING

9.2.1 Description and Applications

Instream mechanical dredging (wet excavation) of contaminated sediments is feasible only for relatively shallow, stagnant flows or for isolated ponds and basins where streambed agitation and excessive turbidity will not cause uncontrolled downstream contamination. For contaminated sediments in deep bodies of water or in those with any appreciable flow, low-turbidity hydraulic dredging operations are required. Low-turbidity dredging is any hydraulic dredging operation that uses special equipment (dredge vessels, pumps) or techniques to minimize the re-suspension of bottom materials and subsequent turbidity that may occur during the operation. Conventional hydraulic dredging may cause excessive agitation and re-suspension of contaminated bottom materials, which decreases sediment removal efficiency and which may lead to downstream transport of contaminated materials, thereby exacerbating the original pollution. Low-turbidity hydraulic dredging systems include small specialty dredge vessels, suction dredging systems, and conventional cutterhead dredges that are modified using special equipment or techniques for turbidity control (National Car Rental System, Inc., 1980).

The Mud Cat dredge uses a slow-speed horizontal auger assembly equipped with a mudshield to increase suction efficiency and reduce turbidity. Operating characteristics of the Mud Cat dredge are discussed in detail in Section 7.2.1. The Mud Cat system is well suited for operation in shallow harbors and basins and small rivers where fine-grained sediments have to be dredged and turbidity is a problem. The Mud Cat dredge was 95 to 99 percent efficient in removing sediments and simulated hazardous materials from impoundment bottoms in field tests conducted for the Environmental Protection Agency, with greater removal efficiencies achieved during backward cuts (Nawrocki, 1976). Somewhat similar to the Mud Cat are the VMI and the Delta mini-dredges.

The waterless dredge has the capability of pumping slurries with a solids content of 30 to 50 percent by weight, with little generation of turbidity. It works best in unconsolidated thixotrophic materials. A bucket wheel type dredge, recently developed by Ellicott Machine Corporation, is capable of digging highly consolidated material and has the ability of controlling the solids content in the slurry stream.

An Italian dredging system, Pneuma, uses two-stage vacuum suction to remove sediments in batches with minimal turbidity. This pumping system is highly applicable in confined areas, where conventional dredge equipment cannot be used (Huston, 1976). Other pump dredging systems that require small dilution volumes and minimize turbidity include the Mud Cat SP-810 pumping platform and the Vaughan Company's Lagoon Pumper, both discussed in greater detail in Section 7.2.1. These systems are applicable for removal of high viscosity materials such as sludges, thick muds and consolidated, fine-grained bottom sediments deposited at depths less than 15 feet.

A Japanese construction firm has developed a dredging system for removal of high-density sludges called the "oozer pump" which may have applications in very deep bodies of water such as large rivers or harbors. This system utilizes vacuum suction and air compression to efficiently remove muddy sediments (silt and clay) and sludges with low turbidity (Nishi, 1976).

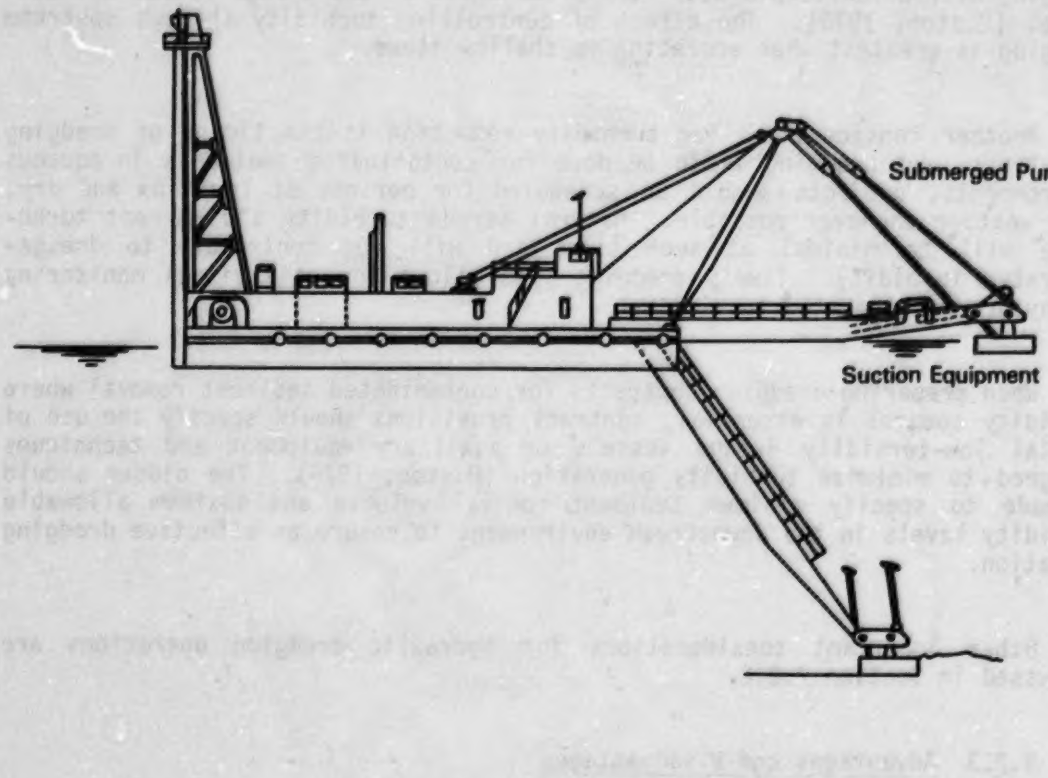
Another Japanese suction dredge, the "Clean Up," (Figure 9-5) uses a hydraulically driven, ladder-mounted submerged centrifugal pump to "vacuum" muddy bottom sediments (fine-grained; high water content) from depths as great as 75 feet, with very low turbidity. This system can pump very dense mixtures (40 to 50 percent solids by volume) at constant flow rates as great as 500,000 gallons per hour, removing up to 900 cubic yards of sediment per hour. A dredge vessel equipped with this pumping system may be used to remove contaminated sediments from large rivers or harbors in depths as shallow as 16 feet, with minimal pollution of the surrounding environment from dredge-generated turbidity (Sato, 1976).

Low-turbidity sediment removal can also be accomplished with conventional cutterhead pipeline dredges (such as the Ellicott Dragon series discussed in Section 7.2.1), if they are modified by cutter removal or the addition of auxiliary pumps. Cutter removal essentially converts the dredge into a suction dredge. With the cutter removed, the suction mouth can be placed directly on the bottom material; this increases suction efficiency, maximizes dredge production, and minimizes turbidity. However, this technique is applicable only for the removal of soft, unconsolidated sediments where cutterhead action is not required to cut and loosen the bottom material. Jets and Ladder-mounted pumps can be installed on conventional cutterhead dredges to increase the head available for lifting bottom material, thereby increasing dredge output and minimizing turbidity. The jet nozzle, usually mounted at or near the suction mouth, increases lifting energy by injecting a high-velocity stream of water into the suction. The use of a jet may increase dredgin efficiency to the extent that cutter removal is feasible (Huston, 1976)

FIGURE 9-5

THE JAPANESE SUCTION DREDGE "CLEAN UP"

(Source: Sato, 1976)


Submerged Pump
Suction Equipment

9.2.2 Design and Construction Considerations

There are certain procedures that all sediment dredging operations may follow to minimize streambed agitation and control turbidity. Where cutterhead dredges are being used, a reduction in the speed of the spiral cutter (in terms of revolutions per minute) will generally result in lower turbidity levels in the immediate vicinity of the cutter (Huston, 1976). Cutter speed reduction may adversely affect dredge production, however, particularly in hardened, irregular sediments.

When dredging in areas of strong currents and natural turbulence, the dredging operation should proceed upstream, into the current, because any turbidity generated must pass around and under the dredge. This will increase the tendency of any suspended material to flocculate and settle. Downstream dredging will allow turbid water to spread ahead of the dredge vessel uninterrupted (Huston, 1976). The effect of controlling turbidity through upstream dredging is greatest when operating in shallow flows.

Another consideration for turbidity reduction is the timing of dredging operations. If dredging is to be done for contaminated sediments in aqueous environments, projects should be scheduled for periods of low flow and dry, calm weather whenever possible. Natural stream turbidity and current turbulence will be minimal at such times and will not contribute to dredge-generated turbidity. Timely dredging also allows for easy visual monitoring of any dredge-generated turbidity.

When preparing dredging contracts for contaminated sediment removal where turbidity control is essential, contract provisions should specify the use of special low-turbidity dredge vessels or auxiliary equipment and techniques designed to minimize turbidity generation (Huston, 1976). The bidder should be made to specify minimum sediment removal volumes and maximum allowable turbidity levels in the downstream environment to ensure an effective dredging operation.

Other important considerations for hydraulic dredging operations are discussed in Section 7.2.2.

9.2.3 Advantages and Disadvantages

Advantages and disadvantages associated with hydraulic dredging techniques are discussed in Section 7.2.3. Low-turbidity hydraulic dredging may entail extra costs when time-consuming modifications are made (such as cutter speed reduction or installation of jets) for purposes of turbidity control. These extra costs are generally balanced, however, by the environmental benefits derived from efficient removal of contaminated sediments.

9.2.4 Costs

Unit costs for hydraulic dredging, including special low-turbidity dredge vessels such as the Mud Cat, have been previously presented in Table 7-8.

9.3 DREDGE SPOIL MANAGEMENT

Contaminated dredge spoil management includes methods for dewatering, transporting, storing, treating, and disposing of contaminated sediments after they have been mechanically or hydraulically dredged from the area of deposition. Related spoil management techniques, which will not be addressed here, include the possible reuse of treated sediments as construction fill in various applications and the treatment of contaminated effluent separated from dredge slurry streams. The most technically and economically effective strategy to handle contaminated sediments removed from a given dredge site will depend on many site-specific variables, which include the following:

- Method of dredging used - hydraulic vs. mechanical
- Method of spoil transport - pipeline vs. truck or hopper barge
- Physical nature of removed spoil - consistency (solids/water content) and grain size distribution
- Volume of removed spoil
- Nature and quantity of contamination; physical and chemical characteristics of contaminant; hazard/toxicity level of contamination
- Proximity of acceptable treatment, storage, or disposal facilities
- Available land area for construction of treatment or containment facilities

There are several well-established techniques for the processing and reuse or disposal of uncontaminated dredge spoil. Techniques for managing contaminated dredge spoil, however, are influenced by the possibly hazardous nature of the spoil material. Special consideration must be given to handling these sediments in a safe, efficient manner. Contaminated spoil treatment and disposal options may be limited because of this fundamental consideration.

9.3.1 Dewatering and Transport

Mechanical dredging generally removes contaminated sediments in discrete, bulk quantities with relatively low water content. Sediments excavated from dry riverbanks, floodplains, or diverted streambeds may be easily truck-loaded without dewatering. For coarse-grained sand and gravel sediments removed by instream mechanical dredging (wet excavation), dewatering is readily accomplished through gravity drainage. Gravity drainage may be performed by dragline or clamshell operators, who simply lift the excavated load and allow any contained water to drain freely over special collection bins. These collection bins are necessary for dewatering of contaminated spoil, since finer-grained contaminated sediments may remain in suspension in the drained water; the bins may be flatbed truck- or trailer-mounted for easy transport to treatment or disposal areas.

Protection of groundwater from contamination by water draining from dredge spoil is a major concern in the handling of this material. Dewatering, storage. and disposal facilities must be designed so as to prevent the infiltration of contaminated water into groundwater. This may involve the use of impermeable liners, underdrains, and collection systems, as described in Chapter 5.

Fine-grained muddy sediments (silt and clay) do not readily drain and generally have a much higher water content than sand and gravel spoil. Because of time considerations, dewatering of these fine-grained sediments generally cannot be accomplished at the dredge site. If their consistency is too dilute, they must be mechanically dredged with special care and truck-loaded for transport to spoil containment areas or processing facilities.

Dewatering of mechanically dredged sediments with a high water content is frequently necessary before the sediments can be economically transported to final disposal areas. Where spoil containment basins are not located at the dredge site and when small quantities of fine-grained spoil are mechanically dredged in dilute form, temporary sand drying beds may be used to dewater the spoil. These are small diked containment areas with a surface layer of 6 to 12 inches of coarse sand underlain by layers of graded gravel. The earthen bottom (preferably of clay) is sloped slightly to vitrified clay tile underdrains placed in trenches. Spoil filling depths are from 2 to 5 feet (Hammer, 1975). Dewatering is accomplished by gravity drainage and air drying (evaporation). The drained water, collected in the tile underdrains, may be gravity-discharged to sanitary sewer lines if sampling indicates low levels of contamination. Alternatively, if high concentrations of a contaminant are detected, the drained water may be collected by sump pumping to portable treatment works. The dewatered soil can be removed from the drying bed by a front-end loader and truck-loaded for transport.

Where large quantities of dilute spoil are mechanically dredged from riverbeds, clamshell or dragline loading onto hopper barges is a transportation option that may be more feasible than truck loading. Hydraulically dredged spoil slurries, usually consisting of about 10 to 30 percent solids (by volume), may also be pumped onto hopper barges for both dewatering and long-distance transport to disposal sites.

9.3.2 Storage and Disposal

Conventional dredged material containment basins serve two basic functions: the removal of dredged solids by sedimentation (settling); and the short-term storage or long-term disposal of these removed solids (Mallory and Nawrocki, 1974). Spoil containment basins can be formed by constructing perimeter berms or dikes around natural topographic depressions. The basins are used to contain spoil slurries pumped through pipelines from hydraulic dredging vessels. Within the basin, sedimentation is the principal process that functions to remove suspended solids from the slurry stream. The surface area and depth of the containment facility, the detention time, the rate at which the dredge pumps into the basin, the solids content of the slurry, and the grain-size distribution of the dredged material are important factors in determining the quantity of solids retained and the resultant effluent water quality (Mallory and Nawrocki, 1974). All these factors, of course, will vary from site to site.

Conventional spoil containment basins are constructed with sluices and overflows to release effluent to natural watercourses in which suspended solids concentrations are low enough to meet state or local water quality criteria. Containment basins designed and constructed to receive spoil slurries containing contaminated sediments must discharge overflow either to sanitary sewer lines (for eventual treatment) or to a secondary containment basin, a clay- or synthetic membrane-lined impoundment with no overflow structures for either permanent or temporary storage.

The required settling area for spoil containment basins is theoretically determined by dividing the settling velocity of the smallest particle to be retained into the basin overflow rate, as previously described in Section 3.4.7 on sedimentation basins. For continuous operations, the basin overflow rate will be equal to the dredge pumping rate. A particle's ideal settling velocity from water is dependent upon the particle's diameter and specific gravity. The real settling velocity of particles is influenced by the temperature and salinity of the water in which they are suspended, the shape of the particles, and the turbulence of flow (Mallory and Nawrocki, 1974).

The settling of fine particles can be somewhat improved with the use of coagulants or polyelectrolytes. Coagulants include metal salts and hydroxides such as ferric sulfate, ferric chloride, and calcium hydroxide. Polyelectrolytes are high molecular weight synthetic polymers that may be either anionic, cationic, or neutral in charge. Polyelectrolytes can be easily injected into the dredge pipeline before the slurry is discharged into the containment basin or barge.

If the dredge site is near a large navigable body of water, it may be possible to employ hopper barges to serve as floating containment basins. Such a system would be especially applicable where land restrictions would prohibit the construction of a containment area. The dredge spoil can be stored and possibly settled if the waterway from which it was removed was relatively quiescent. The injection of a flocculant prior to discharge into the barge would enhance settling. The capacity of the hopper barges and the required amount of settling time would determine the number of barges needed for disposal. A tug boat would also be needed to move barges into position and for transport to a transfer area, where supernatant could be decanted and settled solids could be excavated.

The dredge pumping rate must be adequate to maintain the largest spoil particle present in suspension. Figure 9-6 shows dredging rates typically required to transport various size sediments through different size discharge lines. Figure 9-7 presents typical grain size distributions for dredge sediments of three different classes, based upon the aquatic environment in which they are deposited. The distributions shown are representative of the following situations: Class I, a flowing river; Class II, a harbor with moderate flow or tidal action; and Class III, a relatively still lake (Mallory and Nawrocki, 1974).

When spoil containment facilities attain their maximum storage capacity and they are to be reused, secondary dredging or excavation is necessary to remove the spoil. Contaminated spoil, if it has been sufficiently dewatered, can be mechanically dredged from containment basins with draglines or clamshells and truck-loaded for transport to secured landfills or other suitable disposal areas. Dewatering of contaminated spoil deposited in containment basins may be accomplished through natural drying, the use of sub-spoil sand/gravel drainage layers, vacuum pumping with well-points, or electroosmotic pumping (expensive, but very effective for dewatering fine-grained materials).

For recently deposited spoil with a high water content or for secondary impoundments containing decanted supernatant, hydraulic dredging with a Mud Cat or similar dredge vessel is the most effective means of secondary removal. The dredged spoil would be pumped through overland pipeline to a secondary containment basin for storage permanently or prior to processing facilities.

For mechanically dredged contaminated sediments that are transported in dry, bulk quantities, the most feasible disposal alternative is disposal in a secured landfill -- a facility designed and operated to safely handle wastes that may be toxic, corrosive, ignitable, or chemically unstable. For sediments containing only very low levels of a given contaminant, it may be fea-

FIGURE 9-6

DREDGING RATES TO TRANSPORT VARIOUS MATERIALS

(Source: Mallory and Nawrocki, 1974)

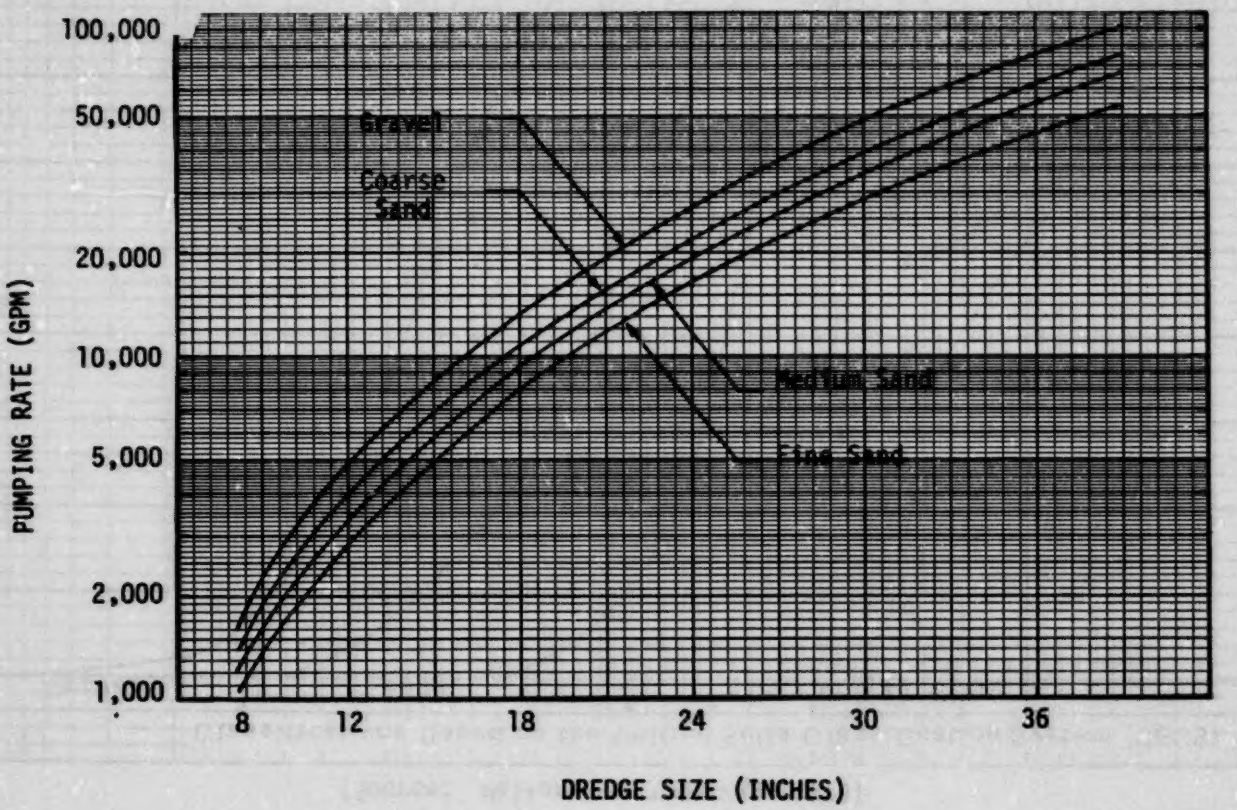
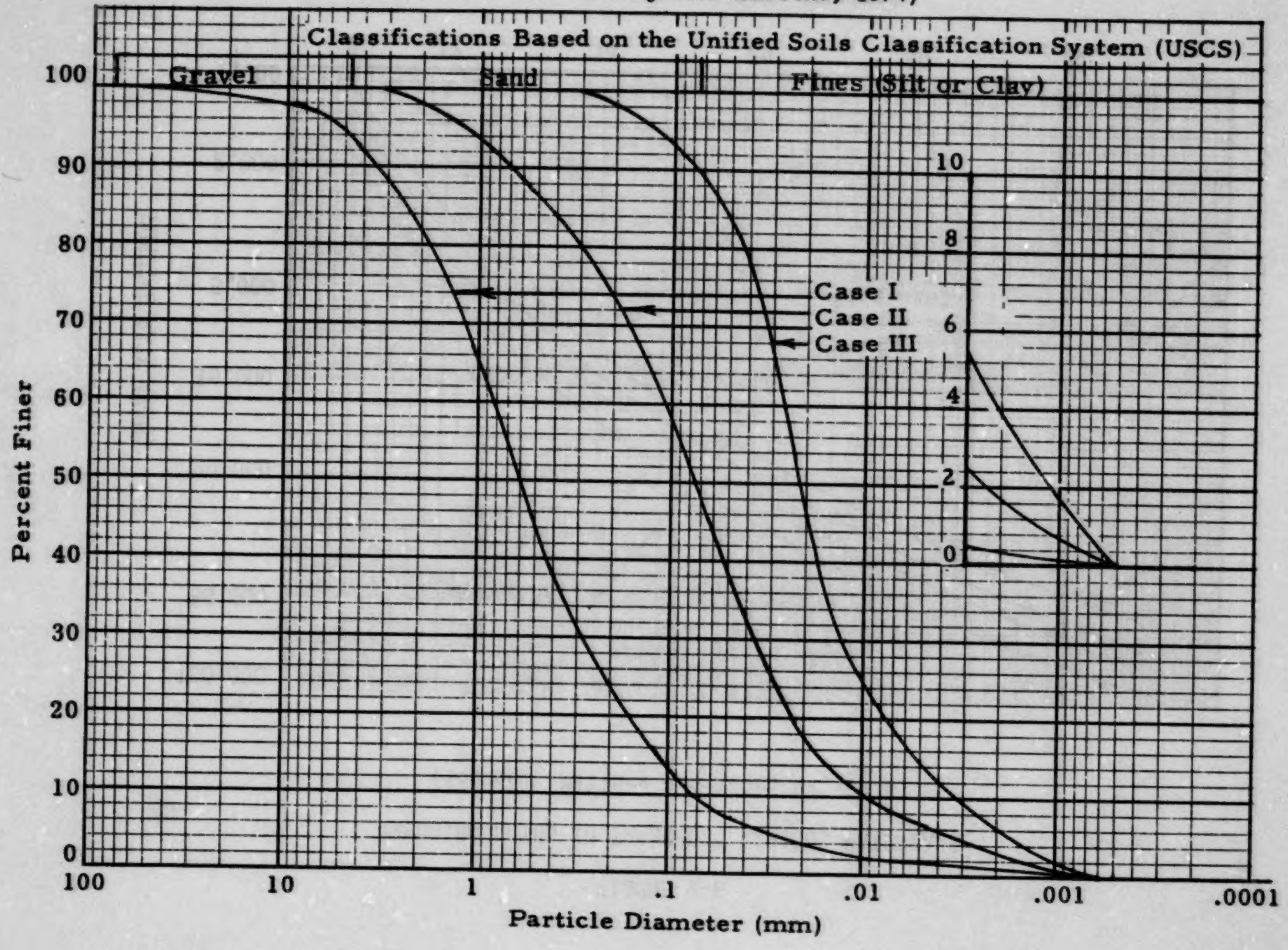


FIGURE 9-7

TYPICAL GRAIN-SIZE DISTRIBUTIONS FOR DREDGED MATERIAL

(Source: Mallory and Nawrocki, 1974)



sible to dispose of mechanically dredged sediments in a conventional municipal solid waste landfill.

9.3.3 Treatment

Physical processing of dredged sediments to selectively remove contaminated particle fractions or to recover uncontaminated fractions from dredge slurry streams may be a feasible spoil management option. Chemical contaminants are oftentimes associated with the fine-grained particles of dredged material; therefore, physical treatment methods such as particle size segregation can serve to isolate contaminated portions of dredged material with a substantial reduction of volume. Processing facilities can be designed that use equipment commercially available in the sand, gravel, and mineral processing industries. These equipment types include hydraulic scalpers, classifiers, thickeners, hydrocyclones, sieves and screens, filters, clarifiers, inclined tube settlers, and flotation separation units. Hydraulic scalpers and classifiers can be used to hydraulically separate sand and gravel (particles greater than 74 microns in diameter) from dredge slurries; when these sediment fractions are uncontaminated, they can be further processed (sized, washed, and blended) to meet construction specifications. The selection of equipment types and combinations for liquid/solids separation, filtration, and sedimentation will depend upon the primary dredge size, the slurry discharge rate, the grain size distribution of the dredge spoil, the total quantity of spoil removed, and the nature of spoil contamination (Mallory and Nawrocki, 1974).

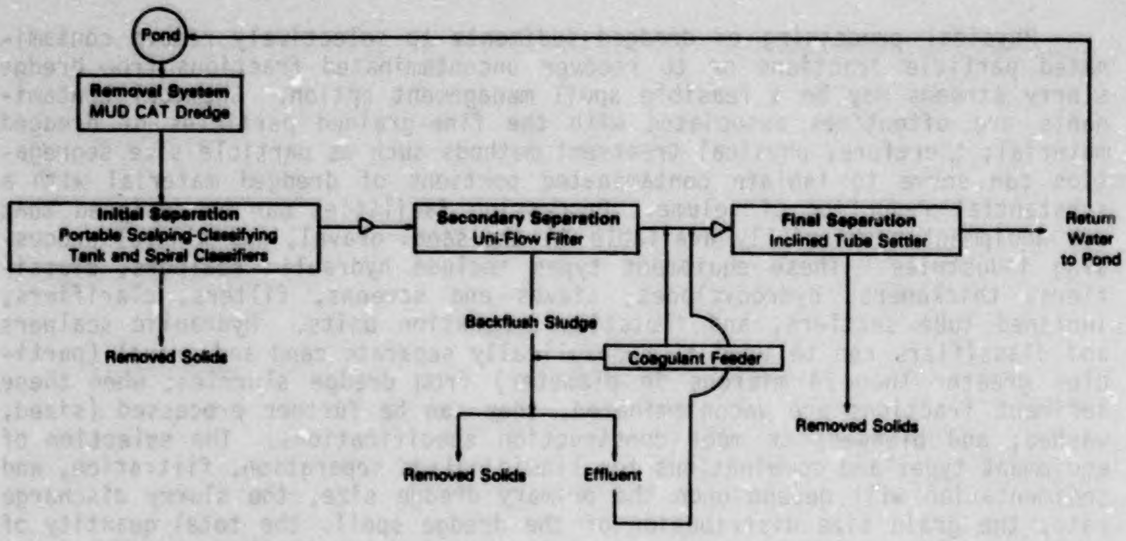
It may be feasible to design and install portable processing systems at the dredging site to remove hazardous particulate material from the dredge discharge stream. Figure 9-8 is a schematic of such a processing system that may be applicable for handling 1500 gpm flows typical of the Mud Cat dredge. Such a system would be totally portable and would be capable of processing a wide range of hazardous materials (in particulate form) hydraulically dredged from a pond or other surface impoundment (Nawrocki, 1976).

In this system, initial solids/water separation is achieved by portable scalping-classifying tanks that hydraulically separate sand-size and coarser particles from the dredge slurry. Spiral classifiers use a large-diameter sand screw to collect, convey, and deposit the removed solids to a discharge pile outside the tank. Overflow from the scalping-classifying tank will go to the secondary separation portion of the system, where removal of fine-grained materials (less than 74 microns in diameter) will be achieved. This may be performed most efficiently with a Uni-Flow filter, a series of banks of hanging polypropylene hoses. The overflow is pumped into and through the hoses, which filter out fine-grained materials in suspension. The sludge that collects on the inside of the hoses is periodically flushed into a collection trough beneath the filter. Chemical coagulants are added to this backflush sludge and the effluent from this process is recycled back into the system for final solids removal. An included tube settler, in conjunction with coagulant

FIGURE 9-8

PORTABLE CONTAMINATED SPOIL PROCESSING SYSTEM

(Source: Nawrocki, 1976)


Pond
Removal System
MUD CAT Dredge
Initial Separation
Portable Scalping-Classifying
Tank and Spiral Classifiers
Secondary Separation
Uni-Flow Filter
Final Separation
Inclined Tube Settler
Return
Water
to Pond
Backflush Sludge
Removed Solids
Coagulant Feeder
Removed Solids
Removed Solids
Effluent

addition to the settler influent, will achieve final separation by removing particles 6 microns in diameter and smaller, ensuring a high quality return flow to the impoundment (Nawrocki, 1976).

Sizing and selection of the individual components of such a processing system should be performed on a site-by-site basis, depending on system flow rate, the grain-size distribution of the solids, and the suspended solids concentration of the dredge slurry. The total costs of such a system would depend on equipment sizes and operating efficiency of the system.

In its simplest form, dredge slurry treatment might consist of adding chemical coagulants (long-chain synthetic polymers or polyelectrolytes) to dredge slurry streams or containment basin materials in order to enhance settling. Coagulation is most effective with very fine clay-sized particles, and it can reduce required containment basin areas by speeding up the sedimentation process (Mallory and Meccia, 1974).

In addition to separating solids from the dredge spoil, it is possible that solids may be further processed to remove the contaminants by leaching with the appropriate organic or inorganic solvent. A hypothetical organic solvent would be hexane, because of its low cost, while an inorganic leaching solution may be either hydrochloric or sulfuric acid (for most metals) or sodium hydroxide (for phenols and some metals). The cleaned sediments may then be disposed of in a normal landfill or possibly returned to their origin.

9.3.4 Design and Cost Considerations

The total costs of a given spoil management strategy must be determined on a case-by-case basis. However, some examples of costs and some general economic considerations are discussed here.

The land area required for construction of conventional spoil containment basins, for settling of both primary and secondary effluent, is an important cost consideration in selecting a method to manage dredge spoil. Where new land must be acquired for containment basin construction, local real estate values will determine initial capital requirements. When existing land disposal sites are located within economic pumping or truck-hauling distances, the major cost consideration will be transportation and handling costs. In general, when fine-grained solids are removed from containment basins by secondary dredging to provide additional spoil storage volume, pipeline transport of a concentrated slurry using booster pumps is less costly than truck loading and transport of the same quantity of dry solids (Mallory and Nawrocki, 1974). Secondary dredging of containment basins, coagulant addition to dredge slurry streams, and sand and gravel removal from slurry streams by processing equipment are three effective methods of reducing required containment basin area, and thereby decreasing capital outlay required for a spoil management project.

If hopper barges are considered as an alternative to containment basins, costs of barge and tug rental for the period of dredging, plus costs for fuel and crew will have to be considered.

When processing equipment is used to recover uncontaminated fractions of sand and gravel from dredged sediments, and the sand and gravel is washed and blended to meet construction specifications, then processing systems will actually help pay for themselves to a limited extent. Processing also serves to reduce required containment basin area and facilitates handling and disposal of dredge spoil through dewatering and classifying sediments. Therefore, when dredged sediments are not severely contaminated or when only particle fractions of a given grain size are contaminated, processing systems are economically more feasible than the exclusive use of conventional containment basins to store or dispose of dredge spoil. Portable processing systems may therefore represent an economically attractive alternative to conventional spoil management by containment basin settling alone.

Processing facilities and/or containment bains should be constructed as close as possible to the dredging operation. However, existing land disposal sites or processing facilities, although they may be located at considerable distances from the dredge site, will generally preclude new construction of such facilities; pipeline pumping or truck hauling costs to existing sites will not be as great as new construction costs, unless the existing facilities are located at very great distances from the dredging site and there are massive quantities of sediments requiring treatment and/or disposal.

Frequently, the most effective spoil management strategy from both a technical and economic viewpoint will combine several of the techniques discussed in this section. An integrated spoil management system might include the following:

- A portable processing system for preliminary sand and gravel removal
- A small containment basin for temporary storage of contaminated fine-grained sediments
- Coagulant addition to the containment basin to enhance settling
- A well point dewatering system to dry the impounded sediments and facilitate mechanical excavation and offsite transport of the sediments to suitable land disposal sites so that the containment basin can be reused
- Secondary hydraulic dredging of the fine-grained sediments and subsequent pipeline pumping to a suitable disposal impoundment (to allow for primary containment basin reuse)

Again, the most cost-effective spoil management strategy will depend on many site-specific variables: the dredge pumping rate; suspended solids content of the dredge slurry; total quantity of solids to be handled; available land area and proximity to dredging site; extent of sediment contamination; form of removed sediments (mechanical dredging - dry solids; hydraulic dredging-dilute slurry); and proximity of existing land disposal or processing facilities.

Some unit costs are presented in Table 9-4, which provides a general picture of the costs associated with dredge spoil management. These data should be used only for conceptual design and planning studies, and equipment manufacturers should be consulted for more detailed cost estimates. Costs for constructing dredge spoil containment basins are presented in Figure 9-9. Excavation and disposal costs can be found in Section 9.1. No cost information was developed for advanced spoil treatment such as solvent extraction. Annual indices presented in Appendix C can be used to update costs based on earlier years.

TABLE 9-4

UNIT COSTS ASSOCIATED WITH DREDGE SPOIL MANAGEMENT

Item	Unit/capacity	Cost	Base year
Portable scalping-classifying tank combined with spiral classifiers	1500 gpm	$84,000	1975[1]
Uni-Flow filter	1500 gpm	$20,000	1975[1]
Hydroclones (5)	1500 gpm	$14,000	1973[2]
Inclined tube setter	1500 gpm	$ 7,000	1975[1]
Coagulant feeder, piping pumps	1500 gpm	$21,000	1975[1]
Low boy trailer	ea.	$12,000	1975[1]
Semi trailers	ea.	$12,000	1975[1]
Land purchase cost (estimated)	acre	$10,000	-
Polymer cost @ 5 ppm	1000 yd^3	$ 12	1979[3]
Tug rental (with crew)	day	$ 3,000	1980[4]
Barge rental (12,000 bbl)	day	$ 300	1980[4]

[1] Nawrocki, 1976
[2] Mallory and Nawrocki, 1974
[3] Betz Laboratories, 1979
[4] C.J. Tippido Co., 1980

Some costing examples using the unit costs presented in Table 9-4 are given below.

* * *

Assume that an area is being dredged at a rate of 1,500 gpm at 20 percent from a contaminated river (Class I sediments). An estimated area of 500,000 square feet must be dredged down about one foot.

Some basic calculations set up the time frame, which in many cases may affect selection of techniques for dredge spoil management:

- Total amount of material to be dredged = 500,000 ft^2 x 10 ft = 500,000 ft^3

- Total dredge discharge to remove sediment at 20% solids (by volume) =

  $$500{,}000\ ft^3 \div 0.20 = 2{,}500{,}000\ ft^3$$

- Total dredging time required =

  $$\times \frac{7.48\ gal}{ft^3} - 1{,}500\ gpm = 12{,}470\ minutes = 210\ hours$$

Assuming a 6-hour working day for actual dredging for 5 days a week (30 hours dredging per week), it would take approximately seven weeks to complete dredging operations. For this relatively small amount of dredge spoil, it may be feasible to construct a zero-discharge lined containment basin. The size requirement for a 10-foot-deep zero-discharge basin would be as follows:

$$Area = 2{,}500{,}000\ ft^3 \div 10\ ft \div 43{,}560\ ft/acre = 6\ acres$$

Costs of a lined based can now be estimated:

- Estimated land cost = 6 acres x \$10,000/acre = \$60,000
- Construction costs (excluding liner) =
  - $6\ acres \times 43{,}560\ \frac{ft^2}{acre} = 261.360\ ft^2$

    From Figure 9.9, the construction cost in 1973 would be about \$40,000.
  - The ENR Construction Index (Appendix C) is used to update costs

    $$\$40{,}000\ (1973) \times \frac{3159}{1895} = \$66{,}700$$
- Costs for clay liner (2 ft thick) at \$8.50 per cubic yard (see Table 3-2).
  - $\$8.50/yd^3 \div 27\ ft^3/yd^3 = \$0.31/ft^3$
  - $261{,}360\ ft^2 \times 2\ ft \times \$0.31/ft^3 = \$162{,}000$

The total cost for a 6-acre clay-lined containment basin is:

$$\$60{,}000 + \$66{,}700 + \$162{,}000 = \$288{,}700$$

An alternative to constructing a sediment basin is to use hopper barges as temporary containment structures. The hopper barge system would be feasible only if a navigable river was located near the site and a feasible treatment scenario could be implemented to dispose of dredge spoil supernatant, i.e., discharge to a sanitary sewer. With a slightly reduced flow of about 14,000 gpm at 6 hours of dredging time per day, one 12,000-barrel-capacity hopper barge would be required to contain one day of dredge output. The required settling time to produce an acceptable supernatant would dictate the total number of hopper barges needed in the system. For the purpose of costing, it is assumed that 5 days are required for settling if flocculants are used. Thus, at least 5 hopper barges will be needed for the containment system, plus a tug for maneuvering and transport. It is assumed that after settling, the supernatant will be pumped to a sanitary sewer and the settled dredge spoil will be draglined into trucks for hauling to a secure landfill. Rental of hopper barges and the tug will depend on projected time to complete dredging. Again, if the total dredge discharge to remove sediment at 20 percent solids is 2,500,000 ft^3, the required time to complete the operation will be as follows:

- Total dredging time =

$$2{,}500{,}000\ ft^3 \times \frac{7.48\ gal}{ft^3} \quad 1400\ gpm = 13{,}360\ minutes = 223\ hours$$

Again, assuming a 30-hour week for actual dredging time, it would take slightly less than 8 weeks for dredging, thus the tug and barges must be rented for 40 days assuming a 5-day work week. Costs for the barge system are as follows:

- Tug rental (with crew) = $\frac{\$3000}{day} \times 40\ days = \$120{,}000$

- Barge rental = 5 barges $\times \frac{\$300}{day} \times 40\ days = \$\ 60{,}000$

- Polymer addition = $\frac{\$12}{1000\ yd} \times \frac{yd^3}{27\ ft^3} \times 2{,}500{,}000\ ft^3 = \$1{,}110$

- Dragline from barge to dumptrucks (from Table 7-6)
  - Assumed sediment specific gravity of 75 lbs/ft^3
  - Dragline excavation =

$$\frac{\$1.39}{yd} \times \frac{yd^3}{17\ ft^3} \times 750{,}000\ ft^3 = \$38{,}600$$

- Hauling and disposal costs
  - Assumed sediment gravity of 75 lbs/ft^3

- Total tonnage =

$$750{,}000 \text{ ft}^3 \times \frac{75 \text{ lbs}}{\text{ft}^3} \times \frac{\text{ton}}{2000 \text{ lbs}} = 28{,}130$$

- Hauling costs (from Section 7.3);

assume 100 miles transport tons =

$$28{,}130 \text{ tons} \times \frac{\text{truckload}}{20 \text{ tons}} \times \frac{\$3}{\text{mile}} \times 100 \text{ miles} = \$422{,}000$$

o Disposal costs (from Section 7.3) =

$$28{,}130 \text{ tons} \times \frac{\$240}{\text{ton}} = \$6{,}751{,}200$$

The total costs for the barge dredge spoil, not including hauling and disposal, are $219,000. The costs including disposal in a secure chemical landfill would bring the costs to well over $7 million.

It is difficult to compare the two options without a more thorough knowledge of the toxicity of the sediment. It may be possible to dispose of sediments in a municipal landfill, which would reduce disposal costs for the barge system significantly. Also, if the wastes are highly toxic, the constructed containment basin would have to be made more secure or the wastes would have to be excavated and placed in a secured landfill, which would significantly increase the costs of this approach.

The above considerations and associated costs demonstrate the enormous problems encountered when dealing with dredging contaminated sediments, since large capital outlays are required to handle large volumes of waste material within a relatively short time. Also, if dredge spoil supernatant is severely contaminated and cannot be discharged to sanitary or industrial sewerage, the on-site treatment of wastewater becomes a major problem. The best way to circumvent these high containment costs is to reduce the spoil volume by, (1) dredging slowly to decrease liquid entrainment; and (2) using one of the unconventional dredging systems that is capable of pumping a high solids stream.

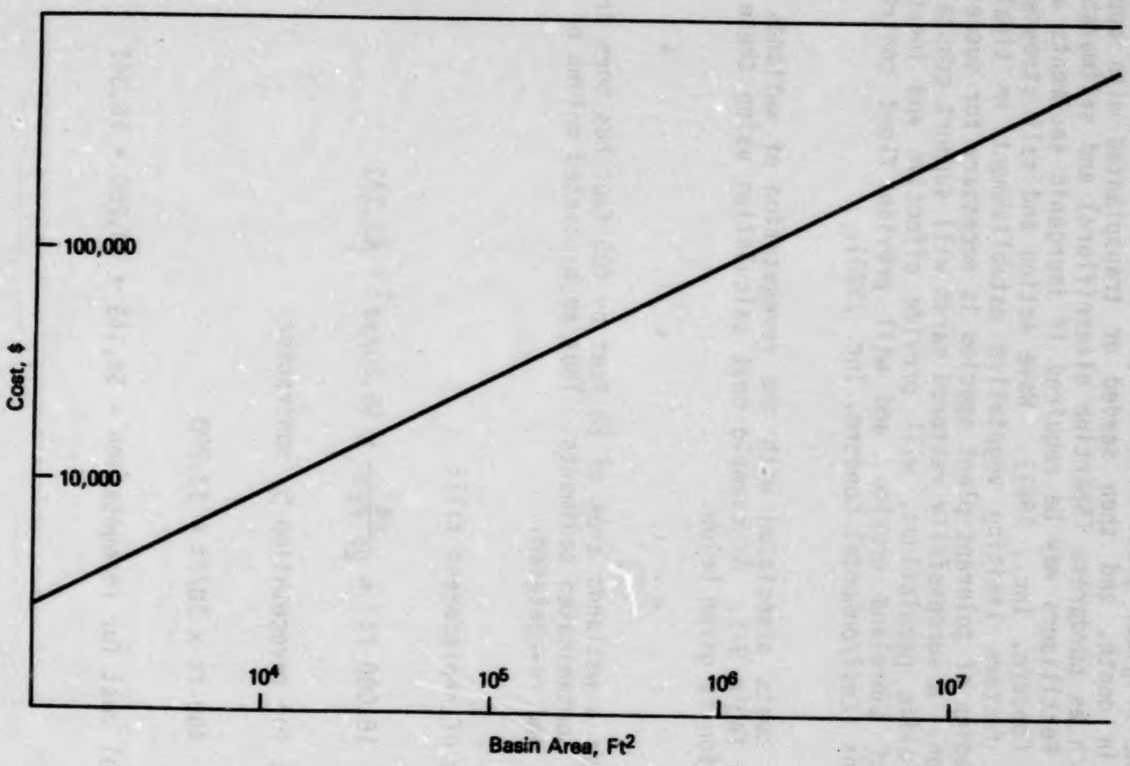
9.4 REVEGETATION

When dredging contaminated sediments in wetlands environments such as tidal marshes and estuaries, an important construction consideration is the possible reclamation of the dredged area with special fill and revegetation techniques. This consideration applies to marshes dredged both hydraulically

FIGURE 9-9

CONSTRUCTION COST OF DREDGE SPOIL CONTAINMENT BASIN

(Source: Mallory and Nawrocki, 1974)



and mechanically. In many instances, a suitable fill material for marshlands is uncontaminated dredge spoil transported from another area of the marsh or from inland sources. Marshland restoration involves subgrading the dredged area with clean fill and resurfacing the area with stockpiled, naturally vegetated marsh materials. Alternatively, the restoration area can be developed to the original grade with inorganic sediments (sands) between one- and two-feet in depth, and then seeded or transplanted with appropriate marsh plants such as cordgrass (Spartina alterniflora) and saltmarsh hay (Spartina patens). Fertilizers may be required if inorganic sediments are used (Environmental Concern, Inc., 1981). Wave action and salt stresses are the most important factors limiting vegetative establishment in tidal marshes, and timely seeding of tolerant plant species is necessary for successful marshland restoration. A successfully restored marsh will support coastal fisheries and other wildlife populations, will provide effective and inexpensive natural control of shoreland erosion, and will provide flood control for coastal floodplains (Environmental Concern, Inc. 1980).

Unit costs associated with the revegetation of wetlands areas are presented in Table 9-5. An example cost calculation using these unit costs for revegetation is given below.

* * *

Assume a wetlands area of 20 feet by 400 feet has been dredged to remove 2 feet of contaminated sediments. Thus an excavated column of 16,000 ft must be filled and revegetated.

Cost of replacement fill:

$$16{,}000\ \text{ft}^3 \times \frac{\text{yd}^3}{27\ \text{ft}^3} \times \$5.30/\text{yd}^3 = \$3{,}141$$

Cost for revegetation by contractor:

400 ft x \$8/ft = \$3,200

Total cost for revegetation = \$3,141 + \$3,200 = \$6,341

TABLE 9-5

UNIT COSTS FOR DREDGED MARSHLAND RESTORATION ACTIVITIES[1]

Description	Unit cost
Plant delivery to restoration site	
● up to 1,500 4-inch peat-potted plants	$0.55/mile (roundtrip)
● up to 5,000 1 3/4 inch peat-potted plants	
● 1,500 to 4,500 4-inch peat potted plants	$0.75/mile (roundtrip)
● 5,000 to 15,000 1 3/4-inch peat-potted plant	
● bare root seedlings, springs and seed	packing and mailing costs FOB St. Michaels, MD
Plant material	
● seeds (viable), up to 20,000 50,000 - 500,000 >500,000	$5.00/1,000 seeds $150/50,000 seeds $1,000/500,000 seeds
● dormant tubers, bulbs and rhizomes	$0.65 - 0.75 each
● dormant sprigs	$0.10 - 0.12 each
● seedlings, bare root, 1-month old	$0.03 - 0.05 each
● seedlings, bare root, 3-month old	$0.13 - 0.15 each
● peat-potted seedlings, 3 to 5 month old	$0.45 - 0.55 each
● peat-potted seedlings, 5-months old	$0.60 - 0.70 each

--continued--

TABLE 9-5 (Continued)

Description	Unit cost
Replacement fill: bank sand hauled 2 miles, placed and spread	$5.30/cubic yard (Godfrey, 1979)
Total marshland seeding or transplanting contractor cost (including initial fertilization)	$6 - 8/linear foot of shoreline (for areas up to 20 feet wide)
Consulting rates; marine biologist, plant ecologist, or environmental scientist	$13/hour

---

[1] Environmental Concern, Inc., 1980.

REFERENCES

ARMCO, 1979

Barnard W. 1978. Prediction and control of dredged material dispersion around dredging and open-water pipeline disposal operations. U.S. Army Engineer Waterways Experiment Station. Vicksburg, Mississippi.

Betz Laboratories, Treuose, PA. June 1979. Personal communication with P. Rogoshewski.

C.J. Tippido Co., Houston, TX. March 1980. Personal communication with P. Rogoshewski.

Cecos International, Inc., Niagara Falls, NY. February/May 1980. Personal communications with S. Paige and P. Rogoshewski.

Environmental Concern, Inc. 1980. Scope of services available. St. Michaels, MD.

Environmental Concern, Inc., St. Michaels, MD. 1981. Personal communication with P. Rogoshewski.

Godfrey, R. (ed.). 1979. Building Construction cost data, 1980. Kingston, MA: Robert Snow Means Co., Inc.

Hammer M. 1975. Water and waste-water technology. New York: John Wiley & Sons, Inc.

Huston J., and W. Huston. 1976. Techniques for reducing turbidity associated with present dredging procedures and operations. Prepared for U.S. Army Engineer Waterways Experiment Station. Vicksburg, MS. Contract no. DACW 39-75-0073.

Kepner Plastic Fabricators, Torros, CA. May 1980. Personal communication with P. Rogoshewski.

Linsley R., and J. Franzini. 1979. Water-resources engineering, 3d ed. New York: McGraw-Hill Book Company.

Mallory C., and R. Meccia. 1974. Concepts for the reclamation of dredged material. Columbia, MD: Hittman Associates, Inc.

Mallory C., and M. Nawrocki. 1974. Containment area facility concepts for dredged material separation, drying, and rehandling. Final report. Prepared by Hittman Associates, Inc. for U.S. Army Engineer Waterways Experiment Station. Vicksburg, MS. Contract report D-74-6.

McMahon, L., and P. Pereira (eds.). 1979. 1980 Dodge guide to public works and heavy construction costs. New York: McGraw-Hill Information Systems.

National Car Rental System, Inc., Mud Cat Division, Fort Lee, NJ. January 1980. Personal communication with P. Rogoshewski.

Nawrocki, M. 1976. Removal and separation of spilled hazardous materials from impoundment bottoms. Prepared under contract no. 68-03-0304 by Hittman Associates, Inc. for U.S. EPA/ORD, Industrial Environmental Research Laboratory. Cincinnati, Ohio. EPA-600/2/76-245.

Nipak, 1980. Sewer Rehabilitation with Nipak Polyetheylene Pipe. Appeared in February 15, 1980 Advertisement.

Nishi, K. 1976. Dredging of high-density sludge using oozer pump. In: Dredging: environmental effects and technology. San Francisco: WODCON Association.

Richardson Engineering Services, Inc. 1980. Process plant construction estimating standards, vol. 1. Solana Beach, CA.

Sato, E. 1976. Application of dredging techniques for environmental problems. In: Dredging: environmental effects and technology. San Francisco: WODCON Association.

Staples, G., JRB Associates, Inc., McLean, VA. April 1980. Personal Communication with P. Le.

APPENDIX A

MONITORING SYSTEMS

INTRODUCTION

Once design and construction of a remedial action has been completed, it will be necessary to monitor the various environmental media (groundwater, air, surface water). The basic objectives of a monitoring system are to:

- Measure the effectiveness of the implementive remedial action by providing long-term verification;
- Act as an "early warning" system for possible breakdown of the remedial action program;
- Protect groundwater and surface water users from potential harm.

The following sections provide a brief description of the considerations necessary to implement monitoring of the various environmental media. It must be emphasized that an all-encompassing description of monitoring programs is not presented here. However, the necessary background and types of considerations are discussed in sufficient detail to provide the user with an understanding of monitoring systems.

GROUNDWATER MONITORING

The major steps necessary in establishing a groundwater sampling network include:

- Establish background
- Well placement

- Well design
- Sampling program
- Laboratory analysis
- Data interpretation

Establishing a background level of groundwater quality is of primary importance in answering the question "what was the groundwater quality before it entered the landfill site," and "what is the groundwater quality now?" The background well can provide information on groundwater contamination that is not attributable to the landfill site. Many times, high concentrations of nutrients, iron, or pH can be linked to agriculture or mining operations that are located in an aquifer recharge zone. It is therefore necessary to establish a solid groundwater quality background data base to ensure that chemical analysis of monitoring wells is interpreted properly.

Well Placement

A typical monitoring well placement scheme is shown in Figure A-1. Well "A" is the background monitoring well and is located far enough upgradient from the site to insure that the landfill will have no effect on the hydraulic condition at the wall.

Well "B" is located on-site, and is placed in a location where migrating contamination can be detected entering the groundwater. The "B" well will also serve as a first indicator of the effectiveness of the remedial action program. If water quality in the monitoring well does not indicate a steady improvement over time (allowing for program stabilization), it will indicate the need for further remedial action considerations. This well must be very carefully constructed and sealed, in order to prevent vertical migration of contaminants down the well casing.

Well "C" is located downgradient from the site, at a position close enough to detect changes in groundwater quality as soon as possible. These wells should also, over a period of time after the "B" wells, show a similar trend of improvement of groundwater quality. These wells should be screened over the entire distance of the aquifer to ensure that a leachate plume is not passing under the well system.

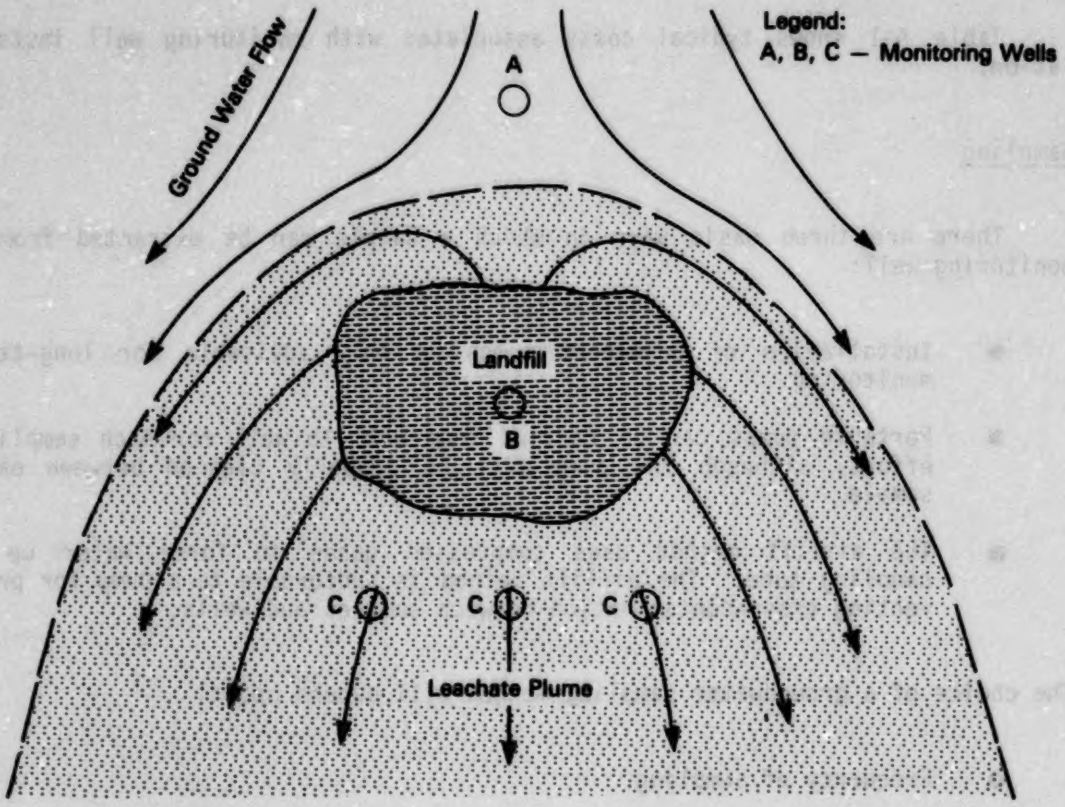
Well Design

The design of the wells installed at the designated locations will depend upon the number and extent of the water-bearing zones to be monitored. If

FIGURE A-1

TYPICAL MONITOR WELL NETWORK, AREAL VIEW

(Source: EPA/530/SW-616)


Legend:
A, B, C — Monitoring Wells
Ground Water Flow
A
Landfill
B
C
C
C
Leachate Plume

only one aquifer system is to be monitored, a single well designed in the configuration shown in Figure A-2 could be used. It is important to note that, if a remedial action program is being monitored for a chemical landfill site that contains organic solvents, PVC casing should not be used. It can be replaced with either case hard steel or plated steel casing.

If more than one aquifer or water bearing zone is to be monitored, a well cluster system will be required at each well location. A typical well cluster system is shown in Figure A-3.

Table A-1 shows typical costs associated with monitoring well installation.

Sampling

There are three basic ways in which a sample can be extracted from a monitoring well:

- Installation of permanent pumps is only advisable for long-term monitoring.
- Portable pumps can be carried from well to well for each sampling effort, although the pump must be properly cleaned between each sample.
- The airlift method uses compressed gases to force water up a sampling tube. The airlift method is preferable to a pump for preventing cross-contamination between samples and wells.

The choice of a groundwater sampling method will depend upon:

- Frequency of sampling
- The number of monitoring wells
- Site-specific conditions

The type of laboratory analyses to be performed for each sample will be dictated by the types of wastes contained at the site. Costs for each sample to be analyzed can range from a few dollars to $5,000 depending upon the number and complexity of the contaminants within each sample.

FIGURE A-2

TYPICAL MONITORING WELL SCREENED OVER A SINGLE VERTICAL INTERVAL

(Source: EPA/530/SW-616)

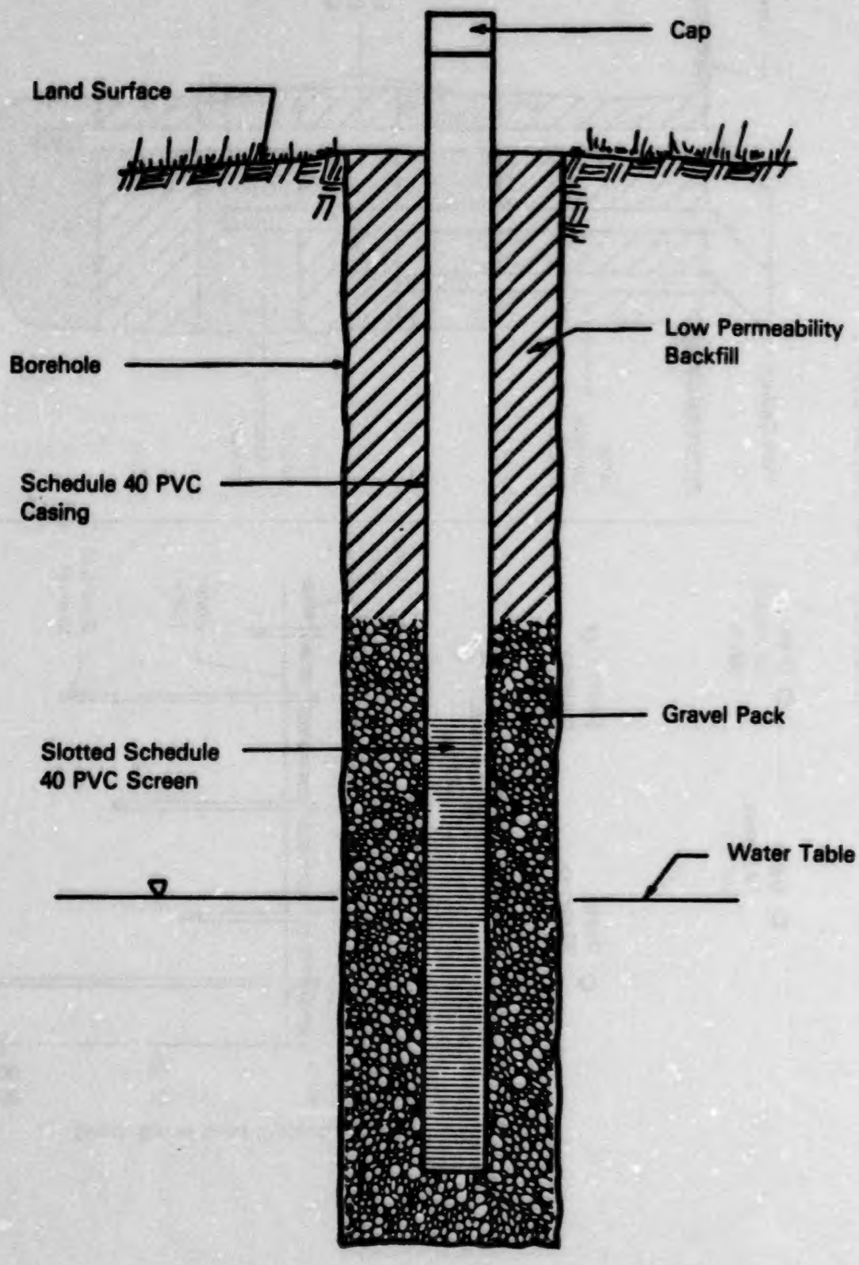
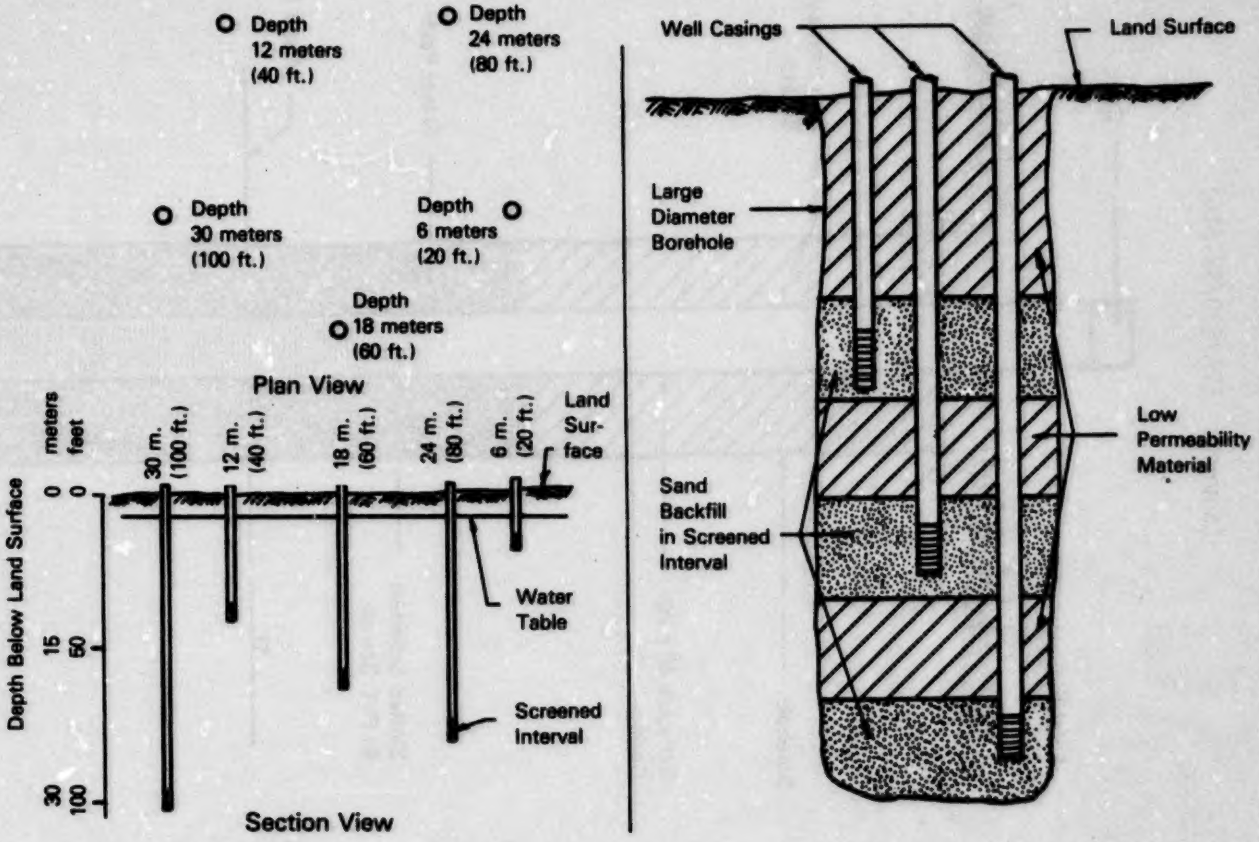

Cap
Land Surface
Low Permeability Backfill
Borehole
Schedule 40 PVC Casing
Gravel Pack
Slotted Schedule 40 PVC Screen
Water Table

FIGURE A-3

TYPICAL WELL CLUSTER CONFIGURATION

(Source: EPA/530/SW-616)



Depth
12 meters
(40 ft.)
Depth
24 meters
(80 ft.)
Depth
30 meters
(100 ft.)
Depth
6 meters
(20 ft.)
Depth
18 meters
(60 ft.)
Plan View
Depth Below Land Surface
meters
feet
0
0
15
60
30
100
30 m.
(100 ft.)
12 m.
(40 ft.)
18 m.
(60 ft.)
24 m.
(80 ft.)
6 m.
(20 ft.)
Land Sur-face
Water Table
Screened Interval
Section View
Well Casings
Land Surface
Large Diameter Borehole
Sand Backfill in Screened Interval
Low Permeability Material

TABLE A-1

COST ESTIMATES FOR VARIOUS MONITORING TECHNIQUES AND CONSTRUCTION METHODS IN THE ZONE OF SATURATION

Monitoring technique & construction method	Price per installation well diameter		
	51mm (2-inch)	102mm (4-inch)	152mm (6-inch)
Screened over a single interval (plastic screen and casing)			
1. Entire aquifer	$1,600-$3,700	$2,300-$4,500	$6,400-$7,500
2. Top 3 meters (10 feet) of aquifer	600- 1,050	700- 1,150	-
3. Top 1.5 meters (5 feet)of aquifer with drive point	100- 200	-	-
Piezometers (plastic screen & casing)			
1. Entire aquifer screened			
a. Cement grout	2,100- 4,700	2,800- 5,500	6,900- 8,500
b. Bentonite seal	1,850- 4,150	2,350- 4,950	6,650- 7,950
2. Top 3 meters (10 feet) of aquifer screened			
a. Cement grout	1,150- 2,050	1,200- 2,150	-
b. Bentonite seal	900- 1,500	950- 1,600	-
Well clusters			
1. Jet-percussion			
a. Five-well cluster, each well with a 6-meter (20-foot) long plastic screen	2,500- 3,800	-	-
b. Five-well cluster, each well with only a 1.5 meter (5-foot) long plastic screen	1,700- 2,300	-	-
2. Augers			
a. Five-well cluster, each well with a 6-meter (20-foot) long stainless steel wire-wrapped screen	4,600- 5,300	-	-

--continued--

TABLE A-1 (Continued)

Monitoring technique & construction method	Price per installation well diameter		
	51mm (2-inch)	102mm (4-inch)	152mm (6-inch)
b. Five-well cluster, each well with only a 1.5 meters (5-foot) long gauze wrapped drive points	1,800- 2,600	-	-
3. Cable tool			
a. Five-well cluster, each well with a 6-meter (20-foot) long stainless steel, wire-wrapped screen.	-	-	9,850-14,150
4. Hydraulic rotary			
a. Five-well cluster, each well with a 6-meter (20-foot) long plastic screen, casing grounted in place.	-	9,050-14,900	13,800-19,400
b. Five-well cluster, completed in a single large-diameter borehole 4.5 meter (15-foot) long plastic screens, 1.5-meter (5-foot) seal between screens.	4,240- 5,800	8,250-11,000	-
5. Single well/multiple sampling point.			
a. 33.5-meters (110-foot) deep well with 1-foot long screens separated by 1.2 meters (4-feet) of casing starting at 3 meters (10-feet) below ground surface.	-	-	3,000- 4,700
Sampling during drilling	-	3,000- 4,700	3,300- 5,200

*Cost estimates are for an aquifer composed of unconsolidated sand with a depth to water of 3 meters (10 feet) and a total saturated thickness of 30 meters (100 feet). Cost estimates are based on rates prevailing in the Northeast in Autumn, 1975. Actual costs will be lower and higher depending upon conditions in other areas. Therefore, while the costs presented will become outdated with time, the relative cost relationships among the monitoring techniques should remain fairly constant.
Source: EPA/530/SW-616

SURFACE WATER MONITORING

If clean-up of a contaminated surface water body is part of the remedial action program, periodic monitoring of the surface water quality will be necessary.

As with groundwater monitoring, the establishing of background water quality is important for determination of the effectiveness of the remedial action program. A location far enough upstream to be unaffected by the landfill site is necessary to ensure proper baseline data quality.

Three stream sampling sites should be adequate to determine the effectiveness of the remedial action program on stream water quality. These are:

- Background upstream
- Closest stream point to landfill site
- Location downstream from landfill site (1/2-mile radius)

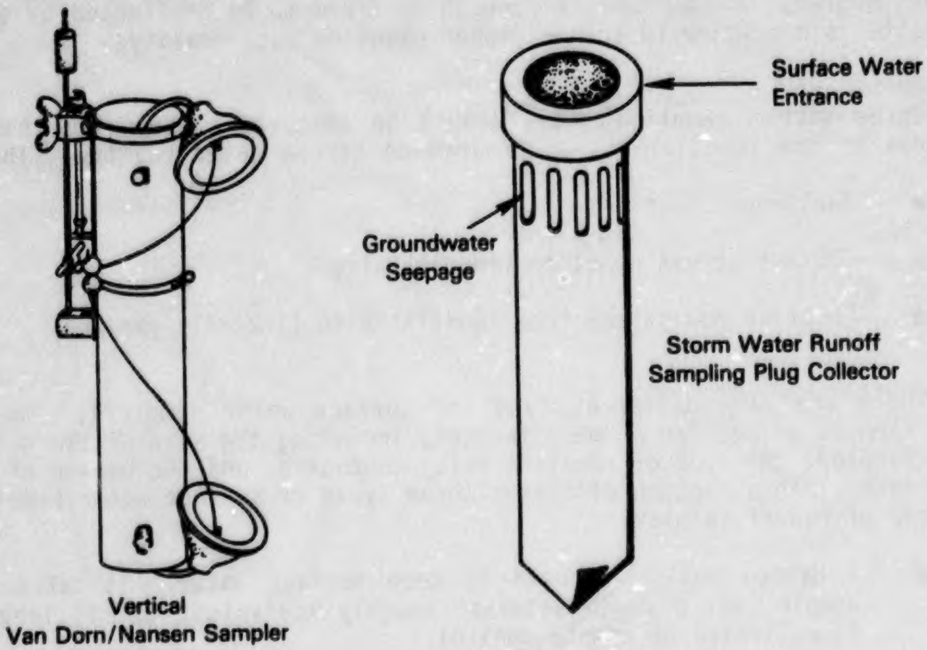
There are many different types of surface water samplers. The choice among them is a function of many factors, including the size of the water body to be sampled, the type of sampling being conducted, and the number of samples to be taken. This section discusses three types of surface water samplers and one type of runoff sampler:

- A Nansen bottle is used in deep surface water. It takes a water sample over a depth interval roughly equivalent to its length, and is activated by remote control.
- The Van Dorn Bottle operates on the same principle as a Nansen bottle, but does not reverse. It is more commonly used because it is less cumbersome. Van Dorn bottles are constructed to take samples in a horizontal or vertical orientation, depending on the depth of the water. A vertical Van Dorn sampler is shown in Figure A-4.
- A grab sampler can be used with a peristaltic pump for sampling at shallow depths. This sample will not allow for specific depth sampling, but is highly effective for quick sampling and is well suited for a preliminary investigation.
- A plus sampler is used for collecting storm water runoff. These samplers should be made of polyethylene, or preferably, Teflon (see Figure A-4). A network of plugs is driven into the ground so that the top face of each plug is just below the surface of the surrounding material. Overland runoff enters each plus through a screen in the top that removes large particles but permits smaller suspended

particulates to enter. These plugs may also be used to collect soil moisture through channels around the side surface by covering the openings in the top faces.

FIGURE A-4

SURFACE WATER SAMPLING EQUIPMENT


Surface Water Entrance
Groundwater Seepage
Storm Water Runoff Sampling Plug Collector
Vertical Van Dorn/Nansen Sampler

GAS MONITORING

Prior to the installation of a gas monitoring system, it will be necessary to determine if the landfill site is capable of generating gases. The type of information needed to make this determination includes:

- Waste type
- Method of disposal
- Age of facility

Landfill facilities that have never received organic wastes will more than likely not be producing gas. Methane gas production requires an anaerobic environment. Therefore, sites that are very young will not have had the time necessary to produce gas. In addition, any sites using a landspreading disposal method will not produce gas.

Volatile toxic substances, however, may produce hazardous gases in relatively young landfills. Reactions between substances in landfills may also form gases that constitute a hazard.

Monitoring Points

It is recommended that there be at least two monitoring points along the property boundaries located in such areas as dry sand or gravel, alignment with an off-site point of concern, proximity of the waste deposit, areas where there is dead or unhealthy vegetation that might be due to gas migration, and areas where underground construction might have created a natural path for gas flow (EPA-SW 828).

On the average, sampling points should be approximately 3 feet in depth. A diagram showing a typical gas probe placement is shown in Figure A-5.

It may be necessary to measure gas concentrations at multiple levels at one point. This can be accomplished by the use of a multi-level gas sampling probe, as shown in Figure A-6.

Sampling of gas is accomplished by using a standard gas monitoring apparatus. Measurements are taken directly from the probes in the field by pumping a sample through the gas explosion meter and obtaining a measurement level.

FIGURE A-5

TYPICAL GAS PROBE PLACEMENT

(Source: EPA-SW 828)

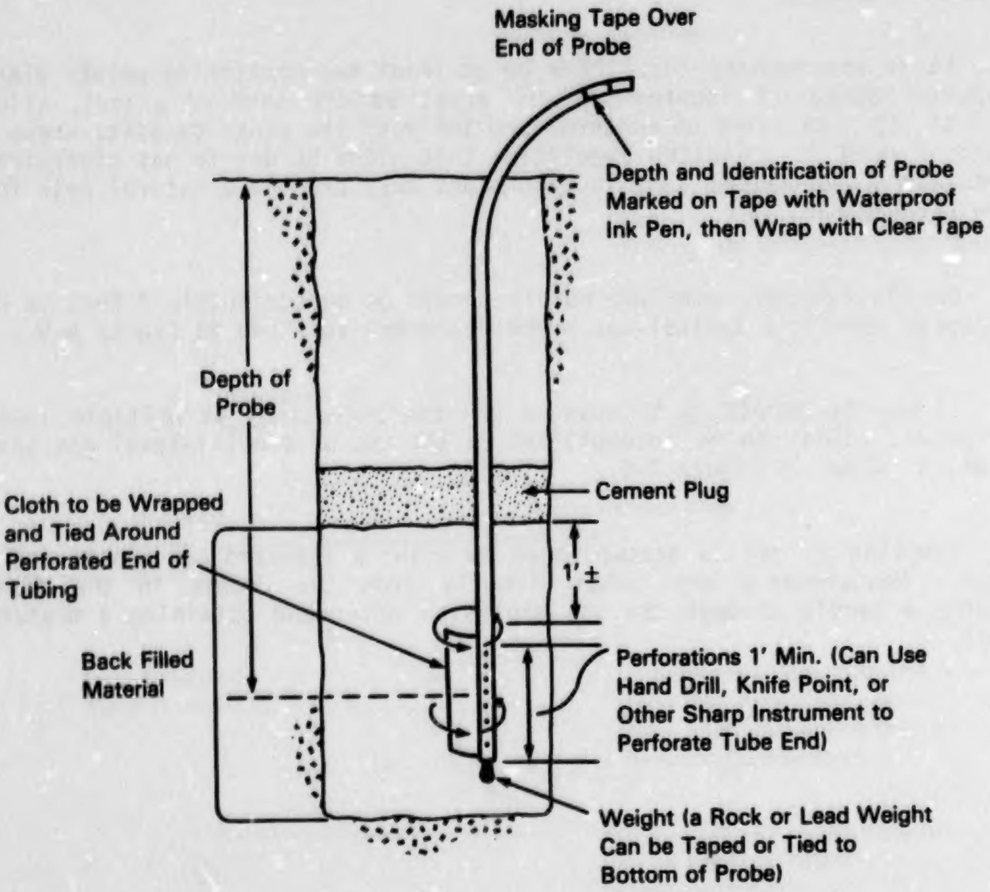
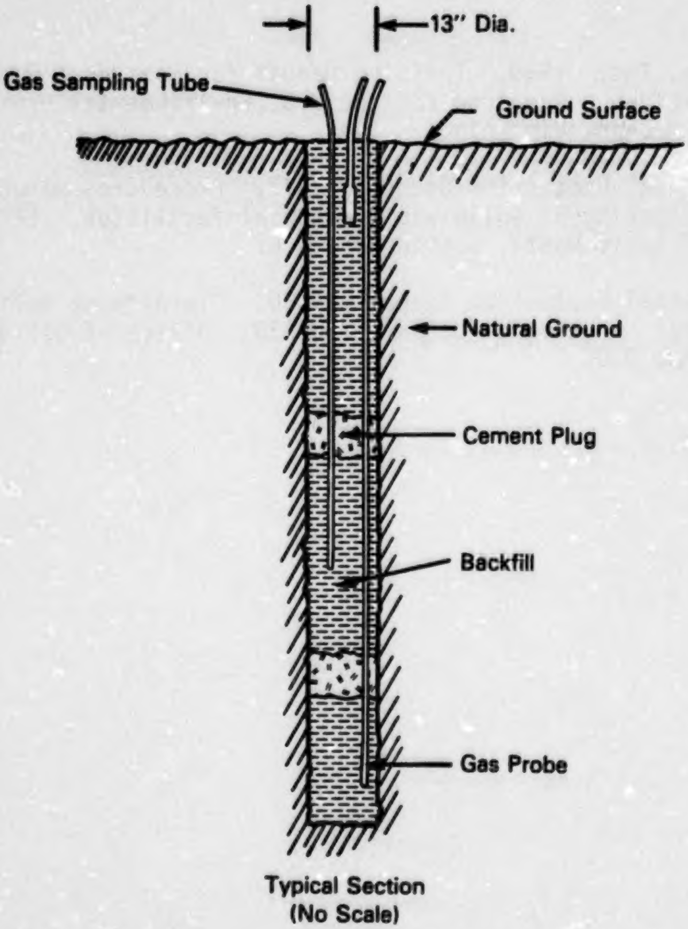

Masking Tape Over End of Probe
Depth and Identification of Probe Marked on Tape with Waterproof Ink Pen, then Wrap with Clear Tape
Depth of Probe
Cement Plug
Cloth to be Wrapped and Tied Around Perforated End of Tubing
1' ±
Back Filled Material
Perforations 1' Min. (Can Use Hand Drill, Knife Point, or Other Sharp Instrument to Perforate Tube End)
Weight (a Rock or Lead Weight Can be Taped or Tied to Bottom of Probe)

FIGURE A-6

TYPICAL MULTI-LEVEL GAS SAMPLING PROBE INSTALLATION

(Source: EPA-SW 828)


13" Dia.
Gas Sampling Tube
Ground Surface
Natural Ground
Cement Plug
Backfill
Gas Probe
Typical Section
(No Scale)

Source: SCS Engineers

APPENDIX A

REFERENCES

JRB Associates, Inc. 1980. Training Manual for Hazardous Waste Site Investigations. Prepared for the U.S. Environmental Protection Agency. McLean, Virginia.

U.S. Environmental Protection Agency. 1977. Procedures manual for groundwater monitoring at solid waste disposal facilities. EPA/530/SW-616. Office of Solid Waste, Washington, D.C.

U.S. Environmental Protection Agency. 1980. Classifying solid waste disposal facilities: a guidance manual. SW-828. Office of Solid Waste, Washington, D.C.

APPENDIX B

WASTEWATER TREATMENT MODULES

This appendix presents a discussion of various treatment modules that may be applicable to leachate treatment. Each treatment module is discussed in terms of applicability to various waste types and strengths. Major design and construction parameters, advantages and disadvantages, and costs are considered. Cost curves were generally prepared to show 1976 costs and may be updated using the table in Appendix C.

The following treatment modules have been considered:

- Flow equalization
- Precipitation, flocculation, and sedimentation
- Biological treatment
  - air-activated sludge
  - pure oxygen-activated sludge
  - trickling filters
  - rotating biological discs
  - biological seeding
  - stabilization ponds/aerated lagoons
- Carbon adsorption
- Ion exchange
- Liquid ion exchange
- Ammonia stripping
- Wet air oxidation
- Chlorination

1.0 FLOW EQUALIZATION

1.1 GENERAL DESCRIPTION AND APPLICATIONS

The primary objective of flow equalization basins is to dampen the flow and concentration fluctuations. Both biological and physical/chemical processes operate more effectively if composition and volume are fairly constant. Because of the high variability in leachate, equalization basins will almost invariably be required to increase the stability of biological and physical/ chemical unit operations.

1.2 DESIGN AND CONSTRUCTION CONSIDERATIONS

In computing equalization volume requirements for leachate treatment systems, it will be necessary to use the water balance equation to determine flow and to design for annual peak rainfall or near peak flow volume of the area. In sizing the equalization basin, the designer will need to determine the amount of fluctuation that the other unit operations in the treatment process can handle without impairing performance, and provide equalization volumes to ensure that fluctuation does not exceed that amount. Equalization basins can be designed for either sideline equalization, where water in excess of the daily flow is equalized, or for in-line equalization where the entire daily flow is equalized. Because of large fluctuations in the concentrations of pollutants, leachate treatment will require inline equalization.

Factors that require consideration in design of the equalization basin include:

- Degree of flow rate and organic loading equalization required to ensure reliable and efficient performance of other process units.
- Aeration and mixing equipment
- Pumping and discharge flow rate control
- Feasible alternative treatment component size for peak flows

Aeration and mixing equipment need to be carefully selected. As a guideline, the minimum mixing required to prevent deposition of solids in municipal treatment systems (at 200 mg/ SS) ranges from about 0.02 to 0.04 hp/1000 gal. Minimum aeration required to prevent septic conditions is about 1.25 to 2.0 cfm/1000 gal.

Because of the large variation in hydraulic head necessary for operation of equalization tanks, pumping is normally required. If a pumping station is required in the headworks, pumps can be designed for the additional head needed for equalization basin operation.

1.3 ADVANTAGES AND DISADVANTAGES

Equalization basins are generally reliable and can be easily designed to achieve the objective. They can dramatically increase the stability of flow and/or concentration of sensitive operations such as carbon adsorption, biological treatment, precipitation, and ion exchange. The only disadvantage is that an equalization basin may require a considerable amount of land area to handle peak flows.

1.4 COSTS

1976 costs for concrete basins with a detention time of one day are shown in Figures B-1 and B-2 (EPA, 1978).

FIGURE B-1

1976 CONSTRUCTION COSTS FOR

CONCRETE EQUALIZATION

BASINS

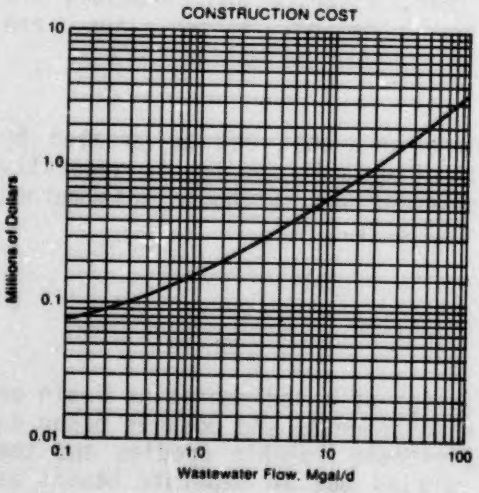
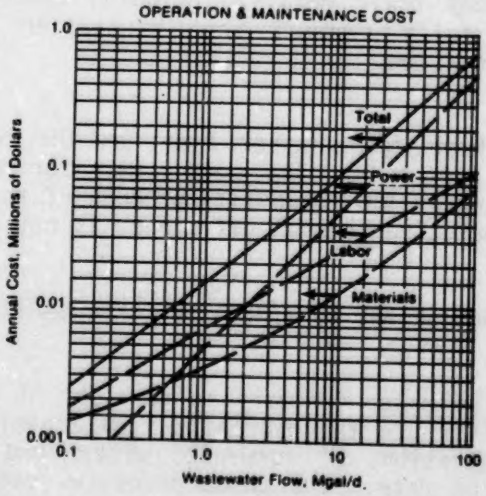


FIGURE B-2

1976 O&M COSTS FOR

CONCRETE EQUALIZATION

BASINS



2.0 PRECIPITATION, FLOCCULATION AND SEDIMENTATION

2.1 GENERAL DESCRIPTION AND APPLICATION

Precipitation, flocculation, and sedimentation are well-developed processes that have been applied to the treatment of various industrial wastewaters containing particulates and/or soluble heavy metals.

Precipitation removes a substance from solution and transforms it into a solid particle. Flocculation promotes particle growth of suspended solids so that they can be more easily removed, and sedimentation removes suspended particles from the liquid.

The processes of precipitation, flocculation, and sedimentation are suitable treatment methods whenever it is necessary to remove precipitable soluble substances and/or suspended solids. The most common applications suitable for hazardous waste sites will include:

- Settling of suspended solids from surface water runoff
- Removal of soluble and insoluble toxic metals
- Removal of soluble inorganics natural to groundwater supplies

Many toxic metals, including cadmium, lead, arsenic, and chromium, are successfully removed from wastewater by precipitation, flocculation, and sedimentation.

There is no upper limit on the concentrations that can be treated by these processes. The lower limit for removal of soluble species is generally governed by the solubility product of the particular ion, although this method of predicting removal efficiency is not very reliable.

2.2 DESIGN AND CONSTRUCTION CONSIDERATION

The major features to consider in the design of a sedimentation basin or clarifier are the hydraulic flow, chemical requirements, and dosages based on concentrations of suspended matter and precipitable soluble species and the settling rate. The three processes can be carried out in separate basins as shown in Figure B-3, or as Figure B-4 illustrates, a clarifier may be used with separate zones for chemical mixing, precipitation, flocculation, and sedimentation.

FIGURE B-3

REPRESENTATIVE CONFIGURATION EMPLOYING PRECIPITATION,
FLOCCULATION, AND SEDIMENTATION

(Source: DeRenzo, 1978)

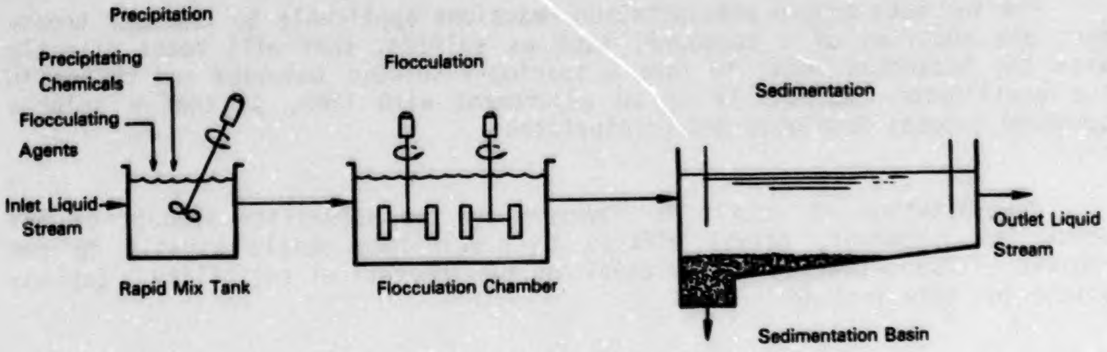
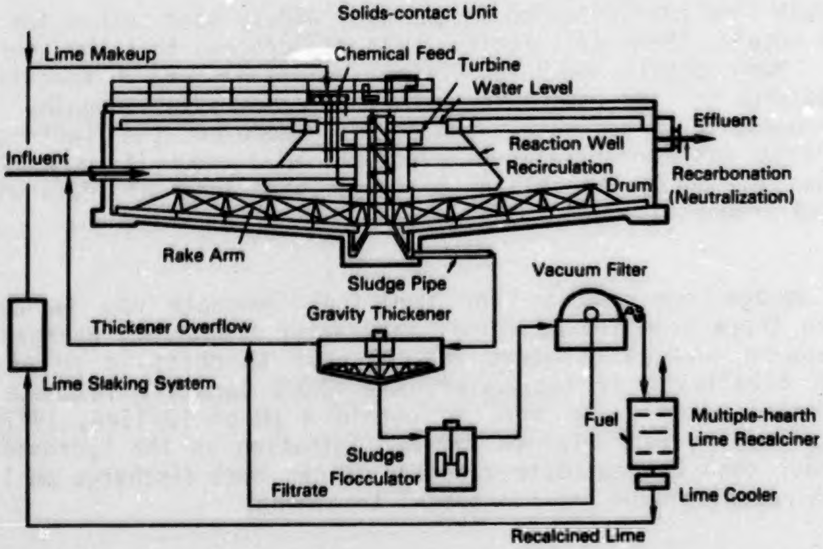

Precipitation
Precipitating Chemicals
Flocculating Agents
Inlet Liquid Stream
Rapid Mix Tank
Flocculation
Flocculation Chamber
Sedimentation
Outlet Liquid Stream
Sedimentation Basin

FIGURE B-4

TYPICAL SOLIDS CONTACT CHEMICAL TREATMENT SYSTEM

(Source: Azad 1976)
See Copyright Notice, Page 497


Solids-contact Unit
Lime Makeup
Chemical Feed
Turbine
Water Level
Effluent
Reaction Well
Recirculation
Drum
Recarbonation (Neutralization)
Influent
Rake Arm
Sludge Pipe
Vacuum Filter
Gravity Thickener
Thickener Overflow
Lime Slaking System
Fuel
Multiple-hearth Lime Recalciner
Sludge Flocculator
Filtrate
Lime Cooler
Recalcined Lime

In applying the processes of precipitation, coagulation, and sedimentation, laboratory tests are available to determine the degree of precipitation along with reaction time and required chemical dosage, the type of flocculant that must be used, and the settling rate.

2.2.1 Precipitation

The two most common precipitation reactions applicable to leachate treatment are addition of a compound, such as sulfide, that will react directly with the hazardous metal to form a sparingly soluble compound and change in the equilibrium, especially by pH adjustment with lime, so that a soluble compound becomes insoluble and precipitates.

Precipitation of metals is governed by the solubility product of the metal ion. However, actual efforts to precipitate metals usually do not achieve effluent concentrations equal to the theoretical solubility. Explanations for this include:

- Many metals form complexes with organo-metallics. These ions are, in some cases, more soluble than the ion itself and may prevent precipitation.
- Cyanide ions or other ions in the wastewater may complex with metals, making them difficult to precipitate as the hydroxide or sulfide (DeRenzo, 1978).

Although lime precipitation is the most widely used method for precipitating heavy metals, there are problems with the process that the user should be aware of. Many metals reach a minimum solubility at a specific pH, but further addition of lime causes the metal to become soluble again. Therefore, the dosage needs to be accurately controlled. However, the fluctuating leachate quantities and concentrations of metals will make it very difficult to control the lime dosage to obtain ideal precipitation; jar tests will need to be conducted frequently.

Lime dosage requirements for municipal leachate may be considerably higher than those used for municipal wastewater treatment; whereas municipal systems require dosages of about 250-400 mg/ℓ to obtain a pH of 10.5, depending on alkalinity of the water, the GROWS landfill leachate treatment system requires about 6,000 mg/ℓ to obtain a pH of 10 (EPA, 1977a). Also, some metals require very high pH for precipitation as the hydroxide, and the effluent must then be neutralized before it can meet discharge pH limitations or be at an acceptable pH for biological treatment.

Precipitation as the metal sulfide is an alternative that has not been used widely. As shown in Table B-1, metal sulfides are less soluble than hydroxides, and generally the metal can be reduced to lower concentrations.

TABLE B-1

APPROXIMATE SOLUBILITIES OF METALS (ppm IN PURE WATER)

Metal	As hydroxide	As sulfide	Factor[1]
Iron	9×10^{-1}	3×10^{-5}	3×10^{4}
Zinc	1×10^{0}	2×10^{-7}	5×10^{6}
Cadmium	2×10^{-5}	7×10^{-10}	3×10^{4}
Nickel	7×10^{-3}	7×10^{-1}	1×10^{5}
Copper	2×10^{-2}	6×10^{-13}	3×10^{15}
Lead	2×10^{0}	4×10^{-5}	5×10^{4}
Mercury	4×10^{-4}	9×10^{-20}	4×10^{15}
Silver	1×10^{1}	7×10^{-12}	1×10^{12}
Chromium	8×10^{-4}	(No precip.)	

$$\text{Factor} = \frac{\text{Solubility as hydroxide}}{\text{Solubility as sulfide}} .$$

[1] Source: Permutit, 1977

2.2.2 Coagulation/Flocculation

Settling of suspended solids depends upon gravitational and/or inertial forces to remove solid particles. Coagulation and flocculation are intended to overcome repulsion forces of individual particles, causing them to agglomerate into larger particles (DeRenzo, 1978). Chemicals used for coagulation and flocculation include alum, ferric chloride, ferric sulfide, lime (coagulants), and polyelectrolytes (flocculants). The effectiveness of a particular coagulant varies in different applications, and in a given application each coagulant has an optimum concentration and pH range. The processes of coagulation and flocculation require rapid mixing followed by a slow and gentle mixing to allow contact between small particles and agglomeration into larger particles. Coagulants must be completely dispersed into water immediately. This is especially true for inorganic coagulants such as alum that precipitate immediately. For lime treatment, it is useful to disperse the lime throughout the wastewater in the presence of recycled sludge to provide an abundance of surface area on which the precipitate can form (Azad, 1976). Rapid mix is usually accomplished in 10-60 seconds. A mean temporal velocity gradient in excess of 300 ft/(sec)(ft) is recommended (Azad, 1976; Liptak, 1974).

The required dosage of coagulant depends upon pH, alkalinity, phosphate levels. and mode of mixing; dosage can be determined by jar tests and zeta potential tests. Typical chemical dosages used in industrial treatment processes are listed in Table B-2. The hydraulic loading, also listed in Table B-2, is used as a basis for determining suspended solids removal efficiencies. The hydraulic loadings shown are intended to achieve 80-90 percent suspended solids removal (Azad, 1976).

TABLE B-2

CHEMICAL TREATMENT OF INDUSTRIAL WASTEWATER BY COAGULATION

Criteria	$FeCl_3$	Alum	$Ca(OH)_2$
Dose, mg/l	80-120	100-150	350-500
Hydraulic loading, gpm/sq ft[1]	0.3-0.4	0.2-0.4	0.5-0.8
Chemical sludge production, lb/million gal	350-700	250-500	4,000-7,000

[1] Without use of polyelectrolytes
Source: Azad, 1976
See Copyright Notice, Page 497.

After achieving effective mix, promotion of particle growth by flocculation is the next step. The addition of flocculants is usually made downstream from the coagulant addition point because the rapid mixing can break up the floc (Liptak, 1974).

Flocculation is accomplished in 15-30 minutes. Mean temporal velocity gradients of 40-80 ft/(sec)(ft) are recommended. The lower value is for fragile floc (aluminum or iron), and the higher value is for lime (Azad, 1976).

2.2.3 Sedimentation

As indicated previously, sedimentation may be carried out in a separate basin from precipitation and coagulation, or all three processes may be carried out in the same basin. When the operations are carried out in combination, two design configurations are available. In the conventional system, rapid mix is completed "in-line" before water enters the large settler where flocculating and clarification are completed. In the sludge-blanket type units, coagulation, mixing, flocculation, and settling all take place in a single unit (Liptak, 1974).

Criteria for sizing settling basins are overflow rate (surface settling rate), tank depth at the side walls, and detention time. For municipal treatment systems, depths average 10 - 12 feet, detention time usually averages 1-3 hours, and surface loading rates average 360-600 gal/d/ft^2 for alum floc, 540-1,200 gal/d/ft^2 for lime floc, and 700-800 gal/d/ft^2 for $FeCl_3$ (EPA, 1978a). Design is considered in detail in Section 3.4.7 for sedimentation basins, and these parameters are generally applicable to clarifiers as well.

In selecting the particular tank shape, proportions, equipment, etc., the designer should:

1. Provide for even inlet flow distribution in a manner that minimizes inlet velocities and short circuiting.

2. Minimize outlet currents and their effects by limiting weir loadings and by proper weir placement.

3. Provide sufficient sludge storage depths to permit desired thickening of sludge. Solids concentrations of 2 to 7 percent should be obtained.

4. Provide sufficient wall height to give a minimum of 18 inches of freeboard.

5. Reduce wind effects on open tanks by providing wind screens and by limiting fetch of wind on tank surface with baffles, weirs or launders.

6. Consider economy of alternative layouts that can be expected to provide equivalent performance.

7. Maintain equal flow to parallel units. This is most important and often forgotten. Equal flow distribution between settling units is generally obtained by designing equal resistances into parallel inlet flow ports or by flow splitting in symmetrical weir chambers (EPA, 1975).

2.2.4 Advantages and Disadvantages

Major advantages and disadvantages of these processes as applied to hazardous wastes are listed in Table B-3.

TABLE B-3

ADVANTAGES AND DISADVANTAGES OF PRECIPITATION, COAGULATION, AND FLOCCULATION

Advantages	Disadvantages
● Process can be economically applied to very large volumes of wastewater	● Process often yields incomplete removal of many hazardous compounds
● Processes have been widely used, equipment is relatively simple	● Large quantities of sludge may be generated
● Processes have very low energy consumption	● Metal precipitate sludges may be an environmental hazard
● There is no upper limit to concentrations that can be treated	● Equipment may be difficult to obtain for flows of less than ~10,000 gpd
	● Because of continually changing leachate quality, required dosages of precipitants and coagulants will continuously change

2.3 Costs

Figure B-5 shows costs for construction, operation, and maintenance of a primary clarifier based on the design criteria listed below.

Costs for chemicals and their storage are not included. However, costs for storage, handling, and feed of lime and alum are shown in Figures B-6 and B-7. Dosages used for these cost estimates are typical for municipal treatment systems but may not be adequate for leachate treatment.

FIGURE B-5

1976 CONSTRUCTION AND O&M COSTS FOR A PRIMARY CLARIFIER

(Source: EPA, 1978)

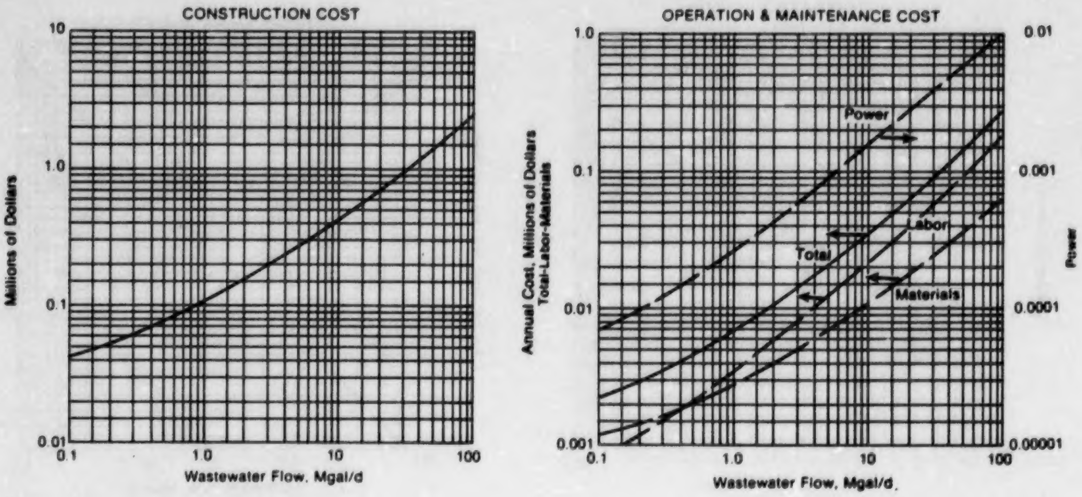


FIGURE B-6

COSTS FOR LIME STORAGE, HANDLING, AND FEED (1976 COSTS)

(Source: EPA, 1978)

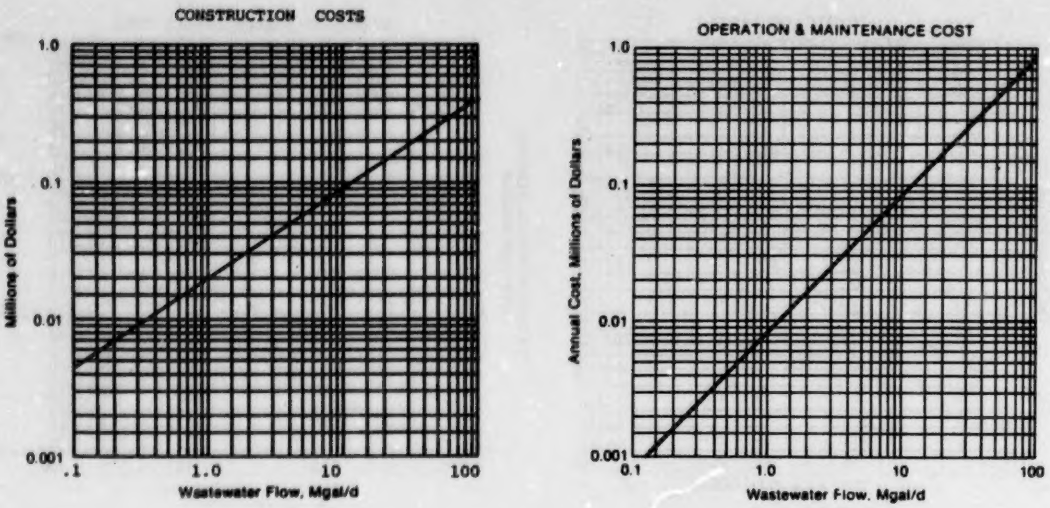
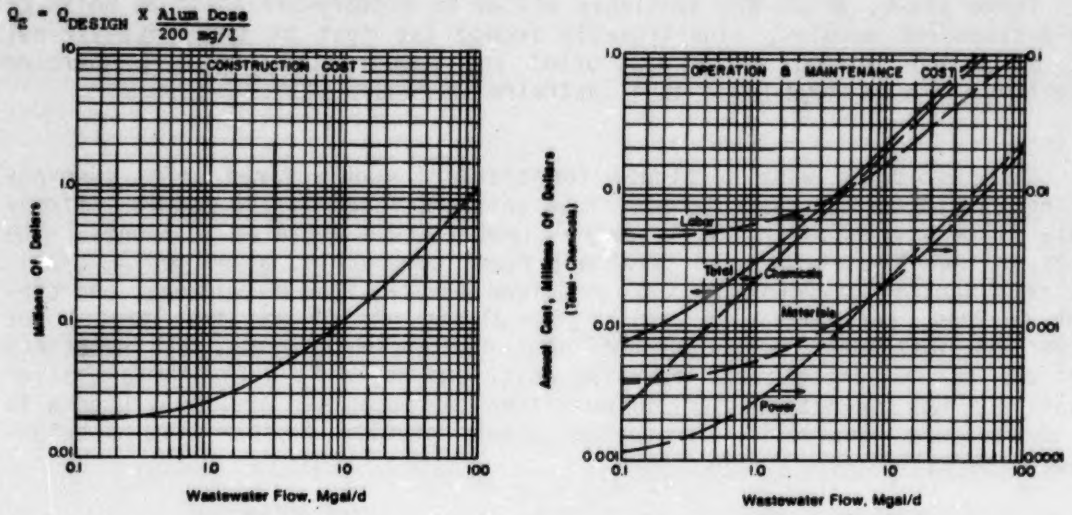


FIGURE B-7

COSTS FOR ALUM STORAGE, HANDLING, AND FEED (1976 COSTS)

(Source: EPA, 1978)



Package plants suitable for coagulation, flocculation, sedimentation, and filtration are available for small flows (10,000 gpd to 2 mgd) as factory-preassembled modules. Construction, operation, and maintenance costs for 1978 are shown in Figures B-8 and B-9. The costs are detailed in Tables B-4 and B-5.

These plans, which are available either as factory-preassembled units or field-assembled modules, significantly reduce the cost of small facilities. The units are automatically controlled and require only minimal operator attention. The package plant is illustrated in Figure B-10.

Cost estimates were developed for standard manufactured units incorporating 20 minutes of flocculation, tube settlers rated at 150 gpd/ft^2, mixed-media filters rated at 2 and 5 gpm/ft^2, and a media depth of 30 inches. The costs include premanufactured treatment plant components, mixed media, chemical feed facilities (storage tanks and feed pumps), flow measurement and control devices, pneumatic air supply (for plants of 200 gpm and larger) for valve and instrument operation, effluent and backwash pumps, all necessary controls for a complete and operable unit, and building. The three smaller plants utilize low-head filter effluent transfer pumps and are to be used with an above-grade clearwell. The larger plants gravity discharge to a below-grade clearwell.

Raw water intake and pumping facilities, clearwell storage, high-service pumping, and sitework, exclusive of foundation preparations, are not included in the costs.

Complete treatment package plants (coagulation, flocculation, sedimentation, and filtration) are designed for essentially unattended operation--that is, they backwash automatically on the basis of headloss or excessive filtered water turbidity, and then return to service (EPA, 1979).

3.0 BIOLOGICAL TREATMENT

3.1 GENERAL DESCRIPTION AND APPLICATIONS

Most organic chemicals are biodegradable, although the relative ease of biodegradation varies widely. With properly acclimated microbial populations, adequate detention time, and equalization to ensure uniform flow, biological treatment can be used to treat a wide variety of organics. There is considerable flexibility in biological treatment because there are a variety of available processes and microorganisms are remarkably flexible. Several generalizations can be made with regard to the ease of treatability of organics by biological treatment.

FIGURE B-8

CONSTRUCTION COST FOR PACKAGE COMPLETE TREATMENT PLANTS AT FILTRATION RATES OF 2 AND 5 gpm/ft (1977 COSTS)

(Source: EPA, 1979)

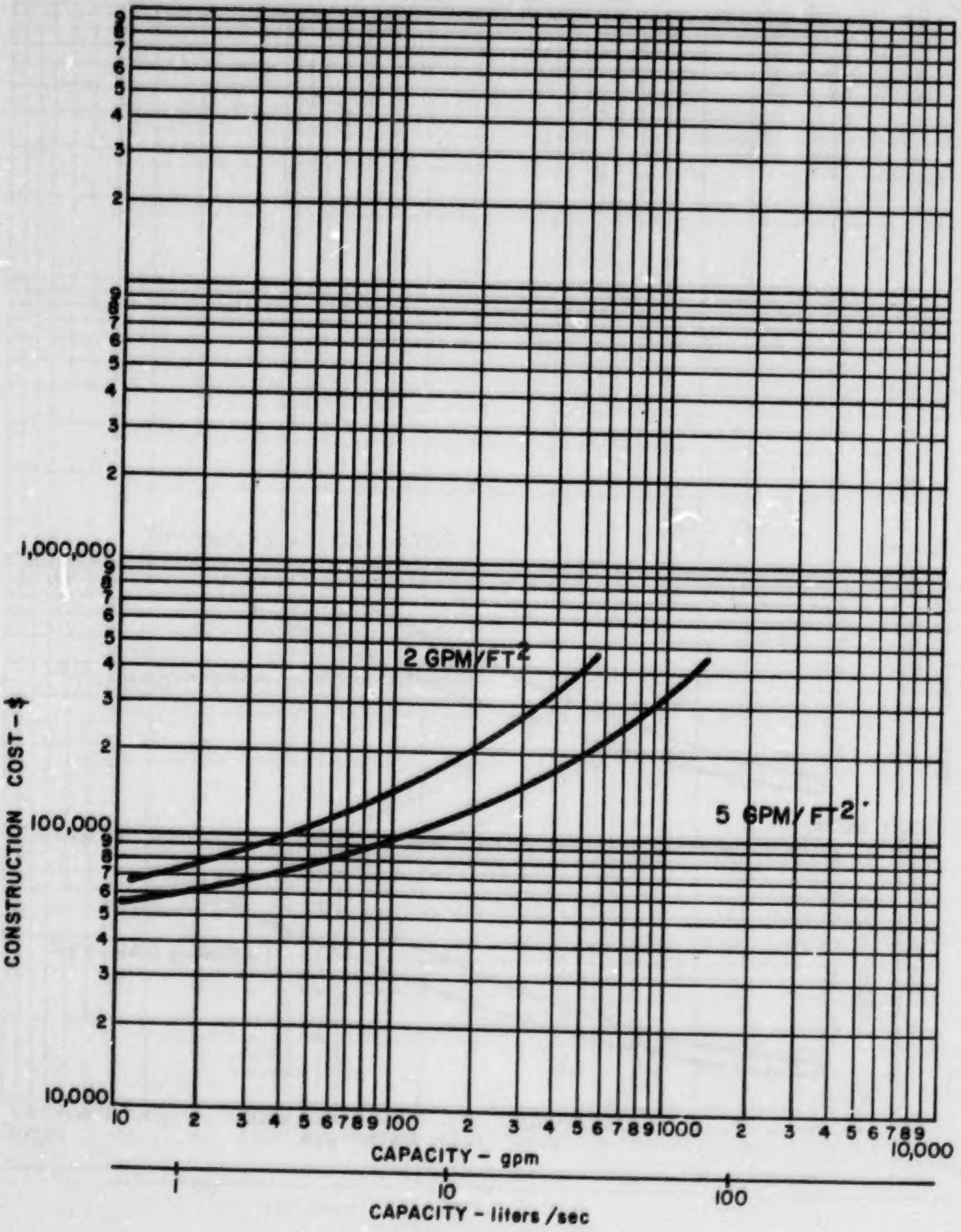


FIGURE B-9

OPERATION AND MAINTENANCE REQUIREMENTS FOR PACKAGE COMPLETE TREATMENT PLANTS - LABOR AND TOTAL COSTS AT FILTRATION RATES OF 2 AND 5 gpm/ft (1977 COSTS)

(Source: EPA, 1979)

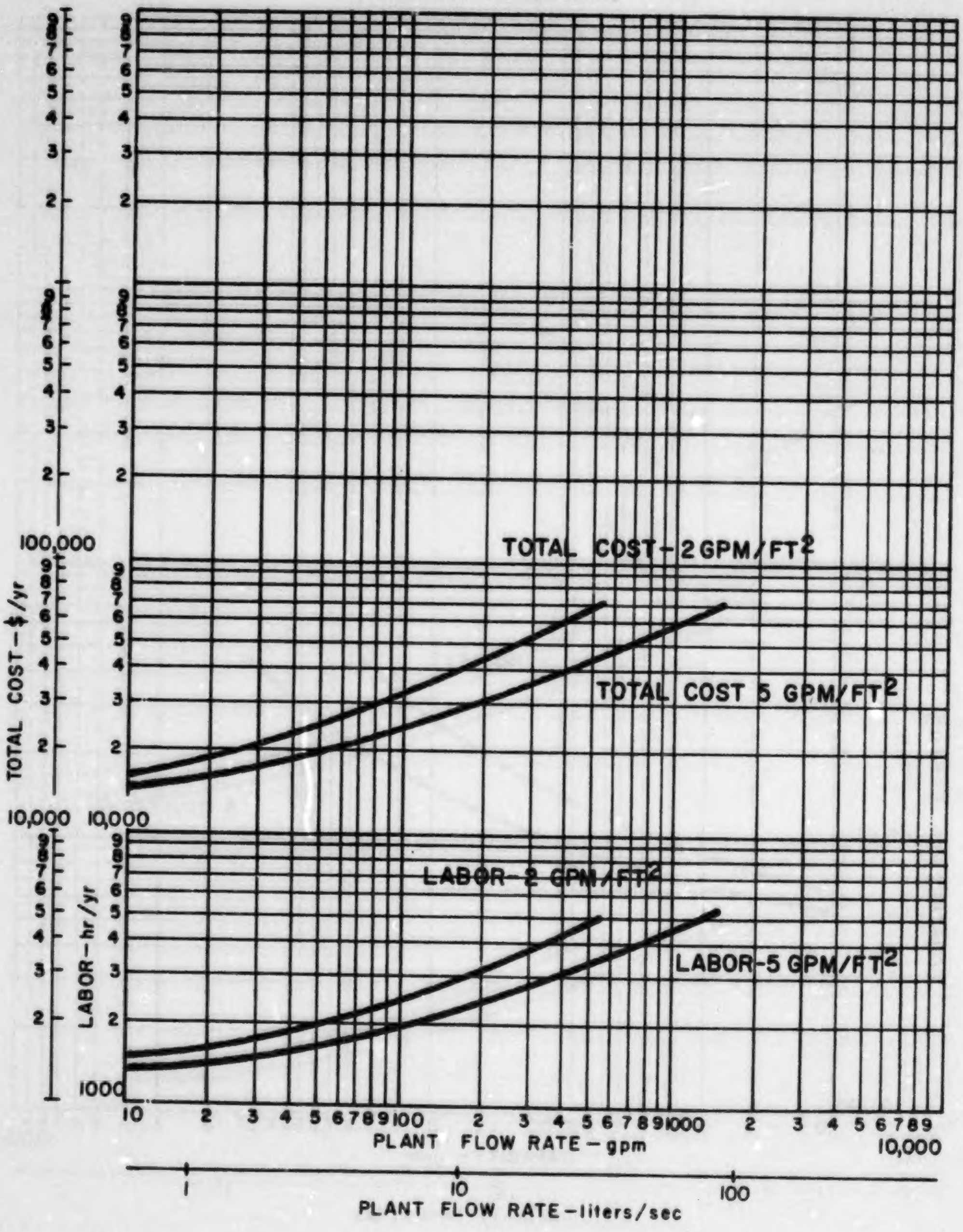
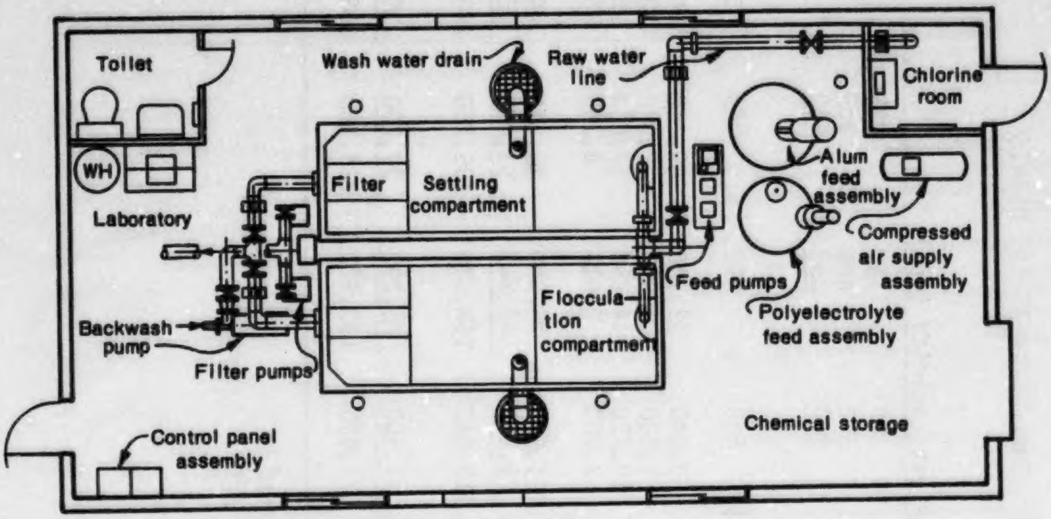


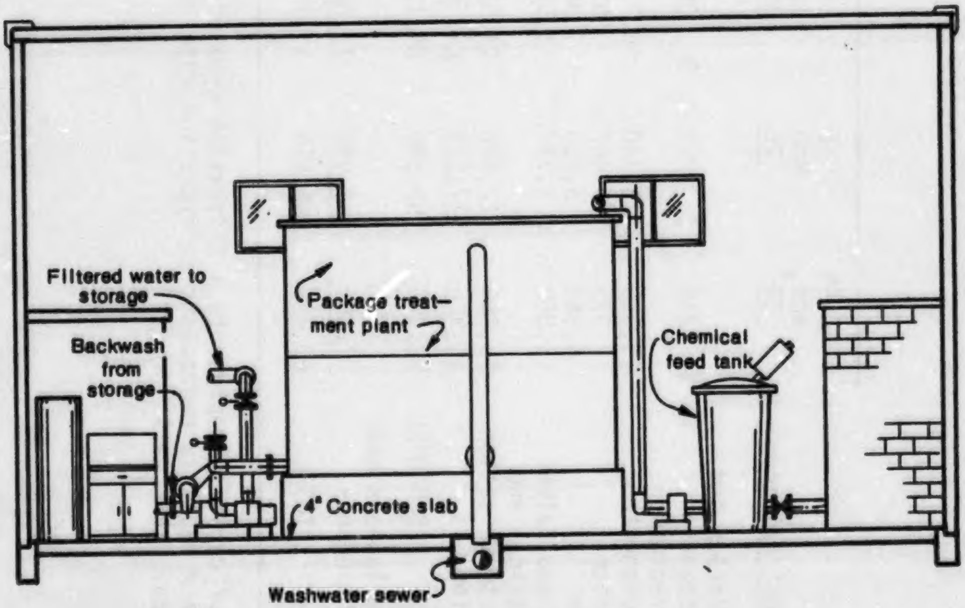
FIGURE B-10

TYPICAL PACKAGE WATER TREATMENT PLANT FOR PRECIPITATION, FLOCCULATION, AND SEDIMENTATION

(Source: EPA, 1979)


Toilet
WH
Laboratory
Wash water drain
Raw water line
Chlorine room
Filter
Settling compartment
Alum feed assembly
Compressed air supply assembly
Feed pumps
Polyelectrolyte feed assembly
Flocculation compartment
Backwash pump
Filter pumps
Control panel assembly
Chemical storage

PLAN VIEW


Filtered water to storage
Backwash from storage
Package treatment plant
Chemical feed tank
4" Concrete slab
Washwater sewer

ELEVATION VIEW

TABLE B-4

CONSTRUCTION COST FOR PACKAGE COMPLETE TREATMENT PLANTS

	Plant capacity (gpm)							
Cost category	4* and 10+	8 and 20	40 and 100	80 and 200	140 and 350	225 and 560	280 and 700	560 and 1,400
Excavation and Sitework	$ 210	$ 270	$ 390	$ 550	$ 810	$ 340	$ 1,310	$ 2,210
Manufactured Equipment	13,050	16,340	30,770	53,040	72,140	89,110	106,090	185,650
Concrete	370	460	690	1,100	1,950	2,070	3,110	4,590
Labor	5,360	6,310	7,360	10,720	14,290	17,660	25,230	39,410
Pipe and Valves	1,060	1,710	1,590	3,980	3,610	4,360	5,950	9.250
Electrical and Instrumentation	16,360	17,570	21,580	21,580	26,990	30,140	49,100	67,210
Housing	15,960	17,460	21,740	29,240	48,840	51,080	73,150	103,880
SUBTOTAL	52,370	59,580	84,120	119,210	168,630	195,260	263,940	412,200
Miscellaneous and Contingency	7,860	8,940	12,620	17,880	25,290	29,290	39,590	61,830
TOTAL	60,230	68,520	96,740	137,090	193,920	224,550	303,530	474,030

*Lower capacity represents a filtration rate of 2 gpm/ft^2
+Higher capacity represents a filtration rate of 5 gpm/ft^2
Source: EPA, 1979

TABLE B-5

OPERATION AND MAINTENANCE SUMMARY FOR PACKAGE COMPLETE TREATMENT PLANTS

	Energy (dw-hr/yr)			Maintenance material ($/yr)	Labor (hr/yr)	Total cost* ($/yr)
Plant capacity	Building	Process	Total			
Filtration Rate of 2.gpm/ft^2:						
4	30,780	320	31,100	$ 320	$1,460	$15,830
8	38,730	390	39,170	590	1,460	16,360
40	61,560	3,210	64,770	860	1,750	20,300
80	98,300	3,950	102,450	1,600	3,200	36,670
140	174,420	6,920	181,340	1,920	3,600	43,360
225	184,680	11,060	195,740	2,140	3,600	44,010
280	277,020	13,830	290,850	2,570	3,600	47,300
560	410,400	27,660	438,060	3,210	5,400	70,350
Filtration Rate of 5 gpm/ft^2:						
10	30,780	780	31,360	320	1,460	15,370
20	38,780	1,360	40,340	590	1,460	16,400
100	61,560	7,810	69,370	860	1,750	20,440
200	98,500	9,470	107,970	1,600	3,200	36,340
350	174,420	16,580	191,000	1,920	3,600	43,650
360	184,680	26,520	211,200	2,140	3,600	44,480
700	277,020	33,150	310,170	2,570	3,600	47,880
1,400	410,400	66,300	476,700	3,210	5,400	71,510

Calculated using $0.03/kw-hr and $10.00/hr of labor.

- Non-aromatics or cyclic hydrocarbons are preferred over aromatics

- Materials with unsaturated bonds such as alkenes are preferred over materials with saturated bonds.

- Stereochemistry affects the susceptibility of certain compounds to attack.

- Soluble organics are usually more readily degraded than insoluble materials. Biological treatment is more efficient in removing dissolved or colloidal materials, which are more readily attacked by enzymes. This is not the case, however, for trickling filters or other fixed media treatment systems, which preferentially treat suspended matter.

- The presence of key functional groups at certain locations can make compounds more or less amenable to degradation. Alcohols, for example, are more easily degraded than their alkane or alkene homologues. On the other hand, addition of a Cl group or an NO_2 group, increases resistance to biodegradation (EPA, 1979b).

Despite the fact that industrial wastes may be refractory to biological treatment for any of the above reasons, microorganisms can be acclimated to degrade many compounds that are initially refractory. Similarly, while heavy metals are inhibitory to biological treatment, the biomass can also be acclimated, within limits, to tolerate higher concentrations of metals. Table B-6 lists concentrations of metals above which treatment efficiency of activated sludge may be inhibited. The remainder of this discussion on biological treatment addresses the general design criteria, advantages and disadvantages, and costs of various biological treatment methods in handling hazardous wastewaters. The treatment methods considered include:

- Air activated sludge and high purity oxygen activated sludge
- Trickling filter
- Rotating biological disc
- Bacterial seeding
- Anaerobic/aerobic and facultative lagoons

TABLE B-6

THRESHOLD CONCENTRATION FOR VARIOUS METALS IN THE AIR-ACTIVATED SLUDGE PROCESS

Metal ion	Concentration (mg/e)	Type activated sludge experiment
Silver	.03	Carbonaceous
Vanadium	10.0	Carbonaceous
Zinc	2.0	Carbonaceous
	5-10	Carbonaceous
	1.0	Nitrification
Nickel	1.0-2.5	Carbonaceous
	1.0	Nitrification
	2.0	Nitrification
Chromium, $^{+6}$	10.0	Carbonaceous
	1.0	Nitrification
	10.0	Nitrification
	1.0	Nitrification
Chromium, $^{+3}$	10.0	Carbonaceous
Lead	10	Carbonaceous
Iron (Ferric)	15	Carbonaceous
Copper	1.0-10.0	Carbonaceous
	1.0	Nitrification
	2.0	Nitrification
Cadmium	1.0	Carbonaceous
	5.0	Nitrification

Source: De Renzo, 1978

3.2 AIR ACTIVATED SLUDGE/PURE OXYGEN ACTIVATED SLUDGE

3.2.1 Applications

The air activated sludge process has been proven effective in the treatment of industrial wastewaters from refineries and coke plants, of pharmaceutical wastes, PVC wastes, and food processing wastes (EPA, 1979; Azad, 1976). Conventional activated sludge has treated petroleum wastes with a BOD_5 as high as 10,000 ppm (Azad, 1976). The process has also been reasonably well demonstrated for the treatment of leachate from municipal landfills. At the GROWS landfill in Bucks County, Pennsylvania, BOD removal of over 98 percent was achieved for an influent concentration of almost 5,000 mg/ℓ.

Treatment included physical/chemical as well as biological treatment (EPA, 1977a). Experiments have shown that activated sludge is generally well suited to treatment of high strength leachates containing high concentrations of fatty acids. As the landfill stabilizes, the ratio of BOD/COD decreases and the wastes become less amenable to biological treatment (EPA, 1977a).

The activated sludge process is sensitive to suspended solids and oil and grease. It is recommended that suspended solids be less than one percent (De Renzo, 1978). Oil and grease must be less than 75 mg/ℓ, and preferably less than 50 mg/ℓ, for effective treatment (Azad, 1976).

3.2.2 Design and Construction Considerations

Key design parameters for activated sludge include (1) aeration period or detention time; (2) BOD loading per unit volume, usually expressed in terms of lb BOD applied per day per 1,000 ft^3 of aeration basin; and (3) the food-to-microorganism ratio (F/M), which expresses BOD loading with regards to microbial mass (mixed liquor volatile suspended solids-MLVSS). There are several modifications of the activated sludge process that may be used depending upon the BOD loading and the required treatment efficiency. Table B-7 summarizes the loading and operational parameters for aeration processes that may be applicable to treatment of hazardous leachate.

Even though conventional treatment has limitations such as a poor tolerance for shock loads, a tendency towards producing bulking sludge that results in high suspended solids in the effluent, and low acceptable BOD loadings, these problems can be alleviated to varying extents with variations in process design. The completely mixed activated sludge process (Table B-7) is the most widely used for treatment of wastewaters with relatively high organic loads. The advantages of this system are:

- Less variation in organic loading, resulting in more uniform oxygen demand and effluent quality
- Dilution of the incoming wastewater into the entire basin, resulting in reduced shock loads
- Use of the entire contactor contents at all times because of complete mixing (Azad, 1976).

The extended aeration process involves long detention times and a low F/M ratio (0.1). Process design at this low F/M ratio results in a high degree of oxidation and a minimum of excess sludge. The contact stabilization process, in which biological solids are contacted with the wastewater for short periods of time, then separated and finally reaerated to degrade sorbed organics, has shown some success for industrial wastes with a high content of suspended and colloidal organics (Ford and Tischler, 1977). Pure oxygen systems have resolved several major drawbacks of conventional treatment. Pure oxygen systems

TABLE B-7

SUMMARY OF OPERATING PARAMETER FOR AIR-ACTIVATED SLUDGE AND PURE OXYGEN-ACTIVATED SLUDGE

Process modification	Aeration system	BOD loading $\frac{\text{LD BOD}^5}{1000\ \text{ft}^3}$	F/M ratio $\frac{\text{LB BOD/day}}{\text{LB MLVSS}}$	MLSS mg/ℓ	Applications and limitations
Conventional	Diffused air, mechanical aerators	20 - 40	0.2 - 0.4	1500 - 3000	Low strength wastes; subject to shock load
Step Aeration	Diffused air	40 - 60	0.2 - 0.4	2000 - 3500	Flexible and generally applicable to a wider range of wastes than conventional treatment. Uses lower volumes of air and shorter detention times than conventional process, but can handle higher BOD loads
Complete-mix	Diffused air, mechanical aerators	50 - 120	0.2 - 0.6	3000 - 6000	Resistant to shock loads, generally applicable
Extended areation	Diffused air, mechanical aerators	10 - 25	0.05 - 2.0	3000 - 6000	Requires long detention times and low organic load produces low volume of sludge; available as package plant
Contact stabilization	Diffused air, mechanical	60 - 75	0.2 - 0.6	1000 - 3000[1] 4000 -10,000[2]	Low aeration requirements; not suitable for soluble BOD

--continued--

TABLE B-7 (Continued)

Process modification	Aeration system	BOD loading LD BOD^5 / 1000 ft^3	F/M ratio LB BOD/day / LB MLVSS	MLSS mg/ℓ	Applications and limitations
High rate	Mechanical aerators	80 up	0.5 - 1.0	4000 -10,000	Well suited to shock loads; requires little supervision. However, requires long detention times, requiring 3 times as much air as conventional treatment
Pure-oxygen	Mechanical aerators	120 up	0.6 - 1.5	6000 - 8000	High efficiency possible at increased BOD loads and reduced aeration

[1] Contact Unit
[2] Solids Stabilization Unit
Source: Hammer, 1975; Metcalf & Eddy, 1972; Nemerow, 1978

show increased bacterial activity, decreased sludge volume, reduced aeration tank volume, and improved sludge settling (EPA, 1979b). The pure oxygen process has been demonstrated to be applicable to a wide range of wastes at high F/M ratios. Such wastestreams include: petrochemical, dye, pharmaceutical, and pesticide wastes (EPA, 1979b).

In addition to process variations, there are several measures available for minimizing process upsets and maximizing stability:

- The deleterious effects of hydraulic and organic load variations can be minimized by equalization preceding biological treatment.
- A commonly used method for providing increased biodegradation is to increase the inventory of biological solids in the aeration basin by increasing the sludge-recycle ratio or reducing sludge wasteage. However, there is usually a trade-off to such an approach. Higher sludge quantities lead to increased need for food and air. Also, old heavy sludge tends to become mineralized and devoid of oxygen, creating a less active floc. The rate of return sludge may vary from 35 to 50 percent in systems carrying a low MISS concentration ($\sim$2,000 mg/ℓ) and from 75 to 100 percent in systems carrying higher MISS (Azad, 1976).
- Suspended solids should be reduced as much as possible by sedimentation or filtration.
- Since kinetics of biological degradation are concentration-dependent, dilution can minimize process upsets under some conditions.
- Sludge bulking, which leads to poor effluent quality, can be controlled by pH control, sufficient aeration, and adequate nutrient supply. An important consideration for leachate treatment is that microbial growth is a function of the limiting nutrient. Some leachates may be phosphorus or nitrogen-limited. Requirements for nitrogen and phosphorus are generally:

  $N = 5$ lb/100 lb BOD_5 removed

  $P = 1$ lb/100 BOD_5 removed

  (Azad, 1976; Hammer, 1975; Metcalf and Eddy, 1972)

Equipment used for activated sludge treatment varies considerably but the major types of aerators are mechanical surface aeration, diffuse air, and sparged turbine aeration.

Mechanical surface aerators are considerably cheaper than diffused aerators, with slow speed mechanical aerators being the cheapest means for oxygenation.

Compressed air diffusion in activated sludge reactors is achieved by two major types of units: fine air diffusers and coarse air bubble diffusers. The operator can increase or decrease oxygenation and mixing by changing the air-blower output. Changes greater than 50 percent are better effected by changing the number of diffusers.

Sparged turbines are mechanically diffused air units. This form of diffused air is very fine and benefits from improved gas transfer kinetics. However, the sparged turbine is generally not as efficient in gas transfer as is the mechanical aerator (Azad, 1976; Metcalf and Eddy, 1972; Hammer, 1975).

Secondary clarifiers are used to separate activated sludge solids from the mixed liquor and to produce concentrated solids for the return flow required to sustain biological treatment. Where multiple tanks are required, it will probably be preferable to use rectangular tanks rather than circular tanks since they require less area. Average hydraulic loading varies from 400 to 800 gal/day/ft^2 and peak loadings range from 700 to 1200 gal/day/ft^2, depending on MLSS concentration and percent sludge recycle. Average solids loading of 0.6 to 1.2 lb/h/ft^2 and peak loadings of 1.25 to 2.0 lb/h/ft^2 are typical for activated sludge plants. Depths are normally 12 to 15 feet.

3.2.3 Advantages and Disadvantages

Advantages and disadvantages of activated sludge and pure oxygen treatment are summarized in Table B-8.

3.2.4 Costs

Treatment costs are dependent upon oxygen requirements, detention time, volumetric loadings, and food-to-microorganisms ratios.

1976 construction and maintenance costs are shown in Figures B-11 and B-12 for conventional treatment using mechanical aeration (EPA, 1978). Costs were based on the following criteria:

Design Basis: ENR Index = 2475

1. Construction cost includes aeration basins. Clarifier and recycle pumps are not included.
2. Volumetric loading = 32 lb BOD_5/d/1,000 ft.
3. 1.1 lb O_2 supplied/lb BOD_5 removed.
4. MLSS = 2,000 mg/l.

TABLE B-8

ADVANTAGES AND DISADVANTAGES OF ACTIVATED SLUDGE AND PURE OXYGEN ACTIVATED SLUDGE

Advantages	Disadvantages
● Activated sludge has been widely used in industrial wastewater treatment	● Capital costs are high
● There are a number of process variations which allow for high degree of flexibility	● Process is sensitive to suspended solids and metals
● Process reliability is good (although not well known for pure oxygen activated sludge)	● Generates sludge which can be high in metals and refractory organics
● Can tolerate higher organic loads than most biological treatment process	● Subject to upsets from shock loads
	● Fairly energy intensive

5. F/M = 0.25 lb BOD_5/d/lb MLSS.

6. Detention time = 6 hours (based on average daily flow).

FIGURE B-11

CONSTRUCTION COSTS FOR CONVENTIONAL ACTIVATED SLUDGE TREATMENT

(1976 COSTS)

(Source: EPA, 1978)

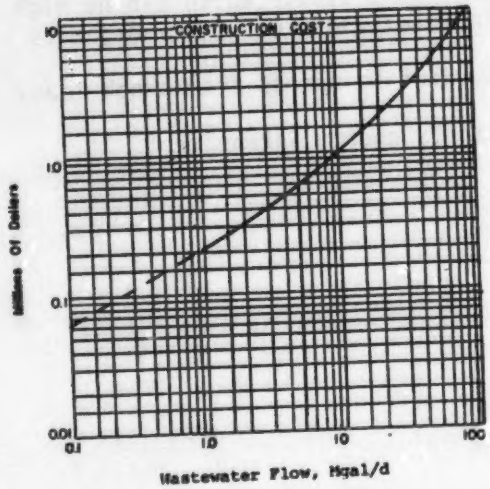
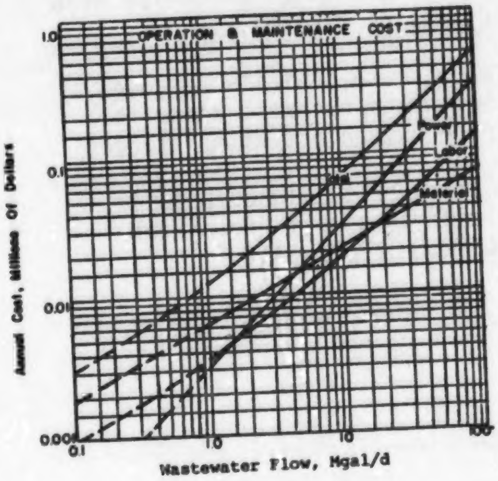


FIGURE B-12

O&M COSTS FOR CONVENTIONAL ACTIVATED SLUDGE TREATMENT

(1976 COSTS)

(Source: EPA, 1978)



Figures B-13 and B-14 show 1976 construction and maintenance costs for extended aeration package plants for low flows of less than 0.1 MGD.

COSTS - Construction cost includes comminutor, aeration basin, clarifier, chlorine contact chamber, aerobic digester, chlorine feed facility, building, fencing for extended aeration package plants between 0.01 and 0.1 Mgal/d. Detention time: 24 hours (based on average daily flow).

FIGURE B-13

CONSTRUCTION COSTS FOR AN
EXTENDED AERATION PACKAGE PLANT
(1976 COSTS)
(Source: EPA, 1978)

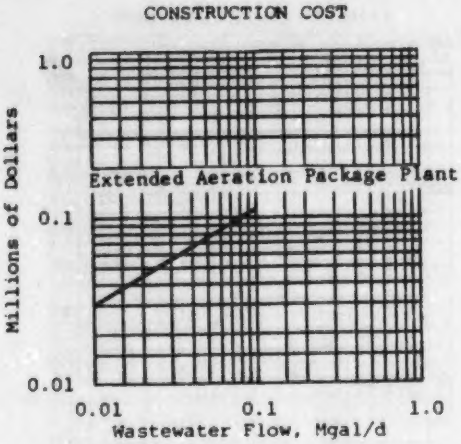
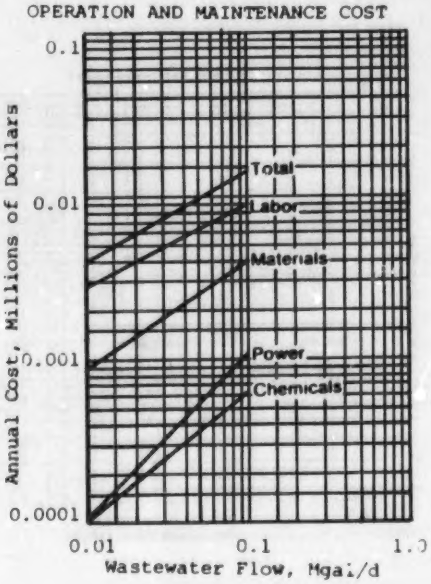


FIGURE B-14

O&M COSTS FOR AN EXTENDED
AERATION PACKAGE PLANT
(Source: EPA, 1978)



Figures B-15 and B.16 shows 1979 construction, maintenance and operating costs for pure oxygen activated sludge (carbonaceous oxidation) based on the following criteria.

COSTS - Construction cost includes oxygenation basins and covers, dissolution equipment, oxygen generators and liquid oxygen feed/storage facilities, instrumentation (where applicable), and licensing fees, ENR Index = 2872. Oxygen was assumed to be delivered as liquid oxygen for plants from 0.1 to 1 Mgal/d size. For plants from 1.0 to 100 Mgal/d, oxygen was assumed to be generated on-site. 1.2 lb O_2 supplier per 1 lb BOD_5 removed. MLVSS = 3,100 mg/l. F/M = 0.5 lb BOD_5/d/lb MLVSS. Detention time = 2 h (based on average daily flow), volumetric loading = 97 lb BOD_5/d/ 100 ft. Oxygen transfer rate (OTR) differs depending upon type of oxygen generator:

- Liquid oxygen generator; OTR = 6.5 O_2/hph
- Pressure swing adsorption (PSA); OTR = 2.0 lb O_2/hph
- Cryogen oxygen generator, OTR = 2.5 lb O_2/hph

FIGURE B-15

CONSTRUCTION COSTS FOR

PURE OXYGEN-ACTIVATED SLUDGE

(Source: EPA, 1978)

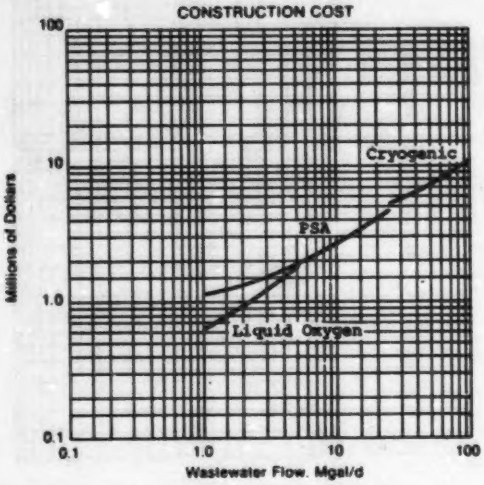
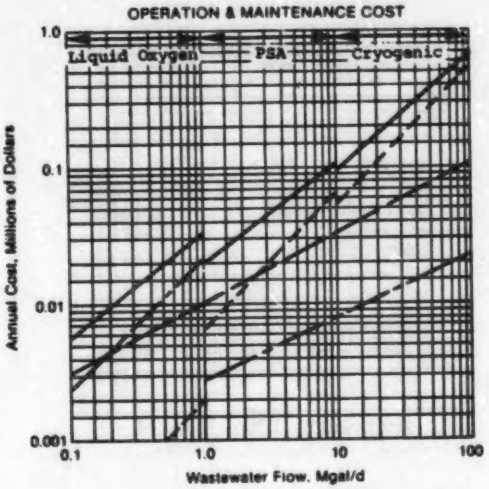


FIGURE B-16

O&M COSTS FOR PURE OXYGEN-

ACTIVATED SLUDGE

(Source: EPA, 1978)



Figures B-17 and B-18 show 1976 construction and O&M costs for rectangular secondary clarifiers based on the following design considerations.

COSTS -

Service Life: 40 years. ENR - 2475 (Sept. 1976), Power Cost: \$0.32/kwn.

1. Flocculator-type clarifier: 600 gal/d/ft^2
2. Costs include sludge return and waste pumps. Sludge concentration of 1 percent solids. Pump TDH at 10 ft. Spare pumps included as necessary. (non-clog centrifugal pumps).
3. To adjust capital cost for alternative flow rates, enter the curve at effective flow (Q_2) = Q_{DESIGN} x 400 gal/d/ft^2 x (1/New Design Overflow Rate).

FIGURE B-17

CONSTRUCTION COSTS FOR

RECTANGULAR CLARIFIERS

SECONDARY CLARIFIERS

(1976 COSTS)

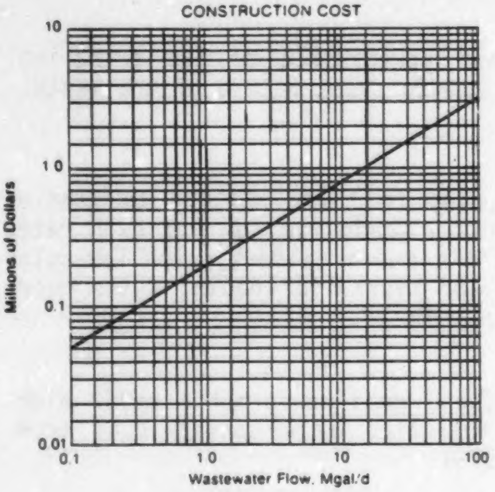
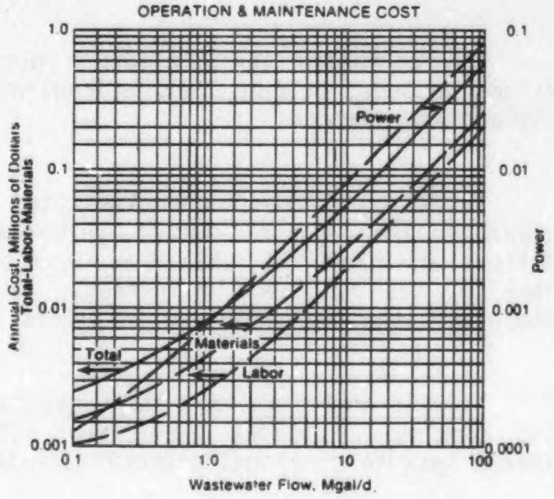


FIGURE B-18

O&M COSTS FOR RECTANGULAR

SECONDARY CLARIFIERS



3.3 TRICKLING FILTERS

3.3.1 Applications

Trickling filters are well suited to treatment of low flow waste streams and are usually used as roughing filters to reduce organic loads to a level suitable for activated sludge treatment. Trickling filters are currently used in conjunction with other treatment methods to treat wastewaters from refineries, pharmaceuticals, pulp and paper mills, etc. (EPA, 1979; Azad, 1976). Efficiency of trickling filters in the treatment of refinery and petrochemical wastes ranges from 10 to 20 percent when used as a roughing filter to 50 to 90 percent when used for secondary treatment (Azad, 1976). The process is more effective for removal of colloidal and suspended materials than it is for removal of soluble matter.

Because of the short hydraulic residence time on the filter material, biodegradation along the filter media is generally insufficient as the sole means of biological treatment. For concentrated wastes, a high rate of recirculation would be required for significant reduction of organics. The short residence time, however, has the advantage of allowing greater variations in influent waste composition as compared to activated sludge or anaerobic digestion. By placing a trickling filter in sequence with activated sludge treatment, the filters could be used to even out loading variations while the activated sludge would achieve the high removal efficiencies needed (De Renzo, 1978; Liptak, 1974).

3.3.2 Design and Construction Considerations

The variables that influence design and performance of the trickling filter include: organic and hydraulic load, media type, nature of the waste, pH, and temperature.

Trickling filters are classified according to their ability to handle hydraulic and organic loads. Typical acceptable loads for low and high rate filters are shown in Table B-9. Use of plastic media filters, with low bulk density, has resulted in increased organic and hydraulic loading rates over those achieved with rock media filters (Table B-9).

Plastic media filters have generally shown good performance under high BOD loading conditions that would not be tolerated by a conventional type system because of clogging problems (Azad, 1976).

Recirculation is generally required to provide uniform hydraulic loading as well as to dilute high strength wastewaters. However, there is a limit to the advantage achievable with recirculation. Generally recirculation rates greater than four times the influent rate do not increase treatment efficiency (Liptak, 1974). Several recirculation patterns are available. One of the most popular is gravity return of the underflow from the final clarifier to a wet well during periods of low flow and direct recirculation by pumping filter discharge back to the influent as shown in Figure B-19.

Several formulas have been proposed which predict BOD removal efficiency based on waste type, influent BOD, hydraulic load and other factors related to performance. Problems with these models include the need to determine treatability on a case by case basis and the fact that the models are usually applicable for only very specific conditions. The reader is referred to articles by Velz, 1960; Germain, 1966; Schulze, 1960; and Eckenfelder, 1963 for these models.

TABLE B-9

DESIGN CRITERIA FOR TRICKLING FILTERS

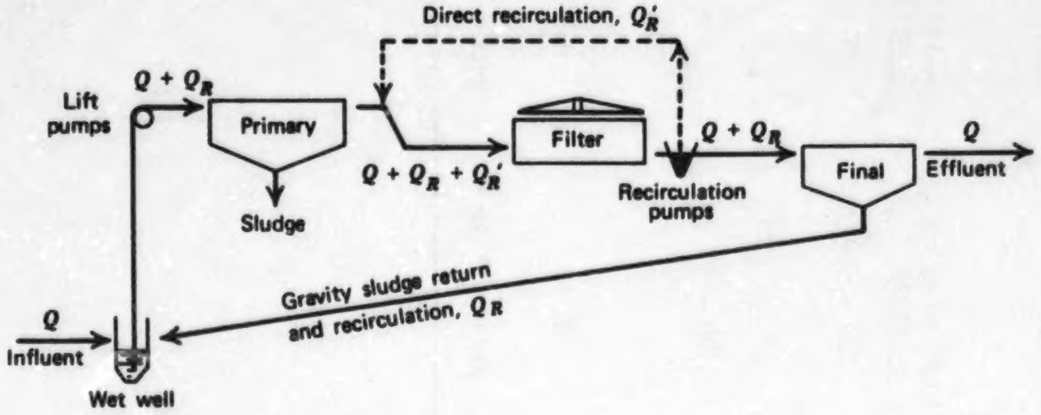
	Plastic media filter	High rate, rock media	Low rate, rock media
Hydraulic loading ($gal/d/ft^2$)	700 - 1400 (Secondary treatment) 2300 - 4600 (Roughing filter)	230 - 900	25 - 90
Organic loading Lbs BOD/day/1000 ft^3	10 - 50 (Secondary) 100 - 500 (Roughing filter)	20 - 60	5 - 20
Bed depth (ft)	20 - 30	3 - 6	5 - 10
Media type	Plastic	Rock - 1" to 5"	Rock - 1" to 5"

Source: EPA, 1978

FIGURE B-19

TRICKLING FILTER RECIRCULATION

(Source: Hammer, 1975)


Direct recirculation, Q_R'
Lift pumps
$Q + Q_R$
Primary
Sludge
$Q + Q_R + Q_R'$
Filter
$Q + Q_R$
Recirculation pumps
Final
Q
Effluent
Gravity sludge return and recirculation, Q_R
Q
Influent
Wet well

3.3.3 Advantages and Disadvantages

Table B-10 summarizes the advantages and disadvantages of trickling filters as compared to other biological treatment methods and non-biological methods for removal of organics.

TABLE B-10

ADVANTAGES AND DISADVANTAGES OF TRICKLING FILTERS

Advantages	Disadvantages
● Because of short hydraulic residence times, process is not highly sensitive to shock loads	● Vulnerable to below freezing temperatures
● Suitable for removal of suspended or colloidal matter	● Limited treatment capability in a single stage operation
● Has good applicability as a roughing filter to even out organic loads	● Potential for odor problem
	● Has limited flexibility and control
	● Requires long recovery time if disrupted

3.3.4 Costs

1977 costs for a high rate trickling filter, rock media plant are shown in Figure B.20 and B.21 for the following design basis.

COSTS Assumptions: ENR = 2494.

1. Construction cost (January 1977 dollars) based on: bed depth = 5 ft; organic loading = 20 lb BOD_5/d/1000 ft^3; recirculation ratio = 4.0 (@ average daily flow) to 0.4 (@ peak daily flow, assumed to equal 3.5 times average daily flow) to maintain average hydraulic loading .75 gal/min/ft^2.

2. Cost includes rock media, underdrains, distributors, and reinforced concrete containment structures. Clarifier and recirculation equipment not included.

3. Operation and maintenance cost includes labor @ $7.50/h and materials. Does not include energy costs.

Water Quality:	Filter Influent (mg/l)	Effluent (mg/l)
BOD_5	130	45
Suspended Solids	100	40

3.4 ROTATING BIOLOGICAL DISCS

3.4.1 Description and Applications

The process is similar to the trickling filter in that the wastes are treated by a fixed-film biological growth. A series of disks are mounted on a horizontal shaft and are place in a countour bottom tank and immersed approximately 40 percent. When rotated out of the tank, the liquid trickles out of the void space and the biomass is aerated. One such process, Biosurf, is illustrated in Figure B-22.

FIGURE B-20

CONSTRUCTION COSTS FOR TRICKLING FILTER PACKAGE PLANT (1976 COSTS)

(Source: EPA, 1978)

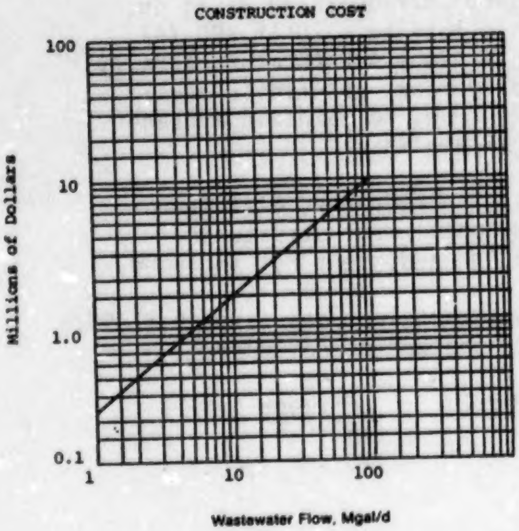


FIGURE B-21

O&M COSTS FOR TRICKLING FILTER PACKAGE PLANT (1976 COSTS)

(Source: EPA, 1978)

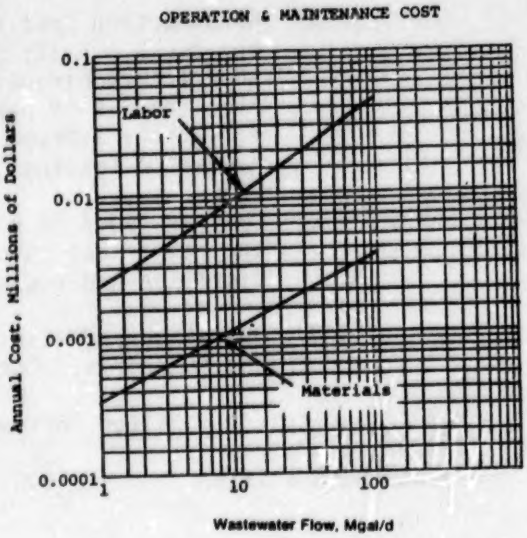
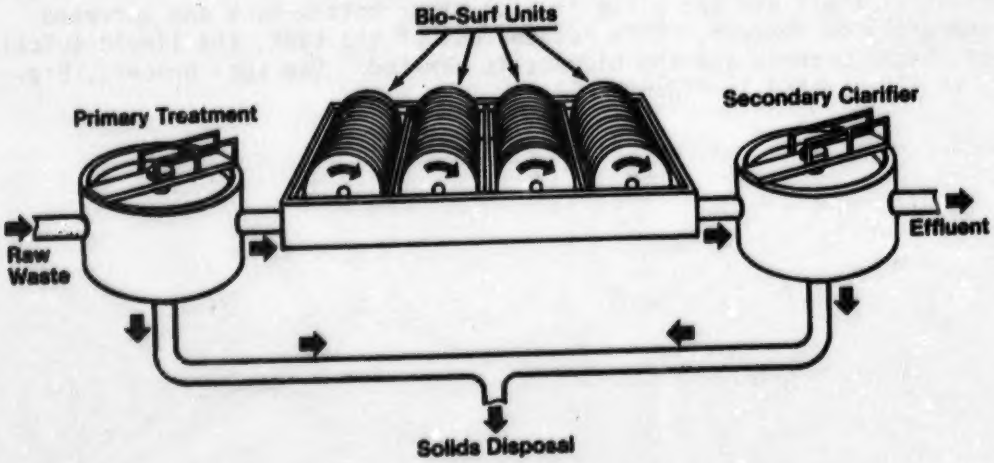


FIGURE B-22

BIO-SURF PROCESS SCHEMATIC

(Source: Autotrol, 1978)


Bio-Surf Units
Primary Treatment
Secondary Clarifier
Raw Waste
Effluent
Solids Disposal

Rotating biological discs are currently being used at full scale to treat wastewaters from the manufacture of herbicides, pharmaceuticals, petroleum, pulp and paper, and pigments (Autotrol, 1978). The process has only been used in the United States since 1969 and is not widespread. However, its modular construction, low hydraulic head loss, and adaptability to existing plants have resulted in growing use (EPA, 1978a). The process can be used for roughing, nitrification, or secondary treatment.

3.4.2 Design and Construction Considerations

For adequate treatment it is recommended that the process include four stages (discs) per train and the use of at least two parallel trains.

Typical design criteria include:

Organic loading: 30 - 60 lbBOD/1,000 ft^3 media (without nitrification)
15 - 20 lbBOD/1,000 ft^3 media (with nitrification)

Hydraulic loading: 0.75 to 1.5 gal/d/ft^2 (without nitrification)
0.3 - 0.6 gal/d/ft^2 (with nitrification)

Detention time: 40 - 90 minutes (without nitrification)
90 - 230 minutes (with nitrification)

(Source: EPA, 1978a)

Based on the design criteria, rotating biological discs can handle organic loads similar to a high rate trickling filter.

3.4.3 Advantages and Disadvantages

Table B-11 summarizes advantages and disadvantages of rotating biological discs as compared to trickling filters and activated sludge.

3.4.4 Costs

1976 costs for rotating biological discs are shown in Figure B-23 and B-24 for flow rates of 0.1 to 10 MGD. Costs are based on the following:

COSTS - (ENR Index = 2475)

1. Construction cost includes RBC shafts (standard media, 100,000 ft^2/shaft, motor drives (5 hp/shaft),

molded fiberglass covers, and reinforced concrete basins.

2. Cost does not include primary or secondary clarifiers.
3. Loading rate - 1.0 gal/d/ft^2.
4. Treatment for carbonaceous oxidation.

TABLE B-11

ADVANTAGES AND DISADVANTAGES OF ROTATING BIOLOGICAL DISCS

Advantages	Disadvantages
● Process has considerably more flexibility than trickling filters; both the intensity of contact between biomass and wastewater and the aeration rate can be easily controlled by the rotational speed of the discs	● Vulnerable to climate changes if not covered
● Wastewater retention time can be controlled by selecting appropriate tank size; thus higher degrees of treatment can be obtained than with trickling filters	● High organic loads may result in first stage septicity and supplemental aeration may be required
● In contrast to the trickling filter, biological discs rarely clog since shearing forces continuously and uniformly strip excess growth	● Odor may be a problem if septic conditions develop
● As compared to activated sludge, rotating biological discs can handle large flow variations and high organic shock loads	● As with trickling filters, biomass will be slow to recover if disrupted
● Modular construction provides flexibility to meet increases or decreased treatment needs	● Can handle only relatively low strength wastes as compared to activated sludge

FIGURE B-23

CONSTRUCTION COSTS FOR ROTATING

BIOLOGICAL DISCS

(Source: EPA, 1978)

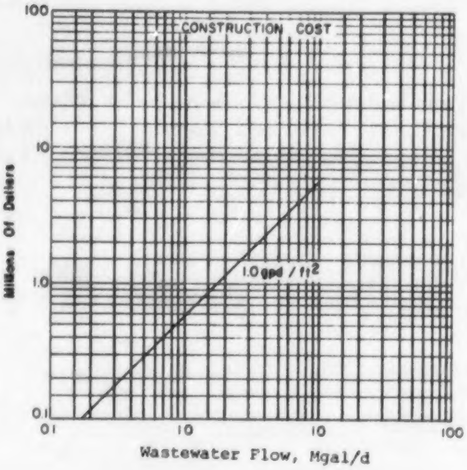
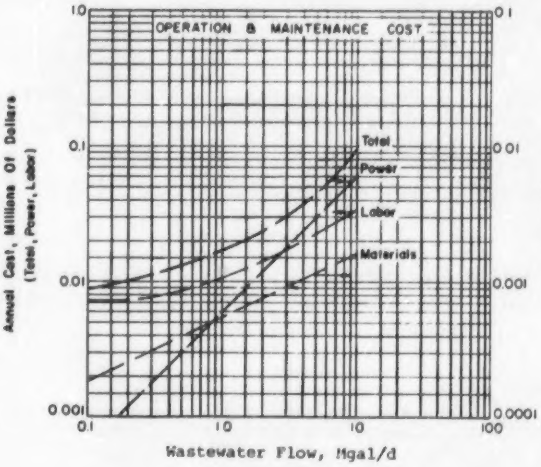


FIGURE B-24

O&M COSTS FOR ROTATING

BIOLOGICAL DISCS

(Source: EPA, 1978)



3.5 BIOLOGICAL SEEDING

3.5.1 Description and Applications

Biological seeding involves addition of acclimated or mutant bacteria that will hasten biodegradation of refractory compounds. As was mentioned previously, it is possible to acclimate bacteria to a wide range of organics. Many organisms are known to induce enzymes needed to degrade "refractory" organics if they are gradually exposed to increasingly higher levels of the "refractory" compound. Thus biomass that has treated petrochemical wastes for a period of time will be more suited to degrading benzenes, naphthalene, and phenols in leachate than would biomass from a municipal treatment process.

The Polybac Corporation offers a freeze-dried, biochemical preparation containing mutant bacteria and substances to enhance their growth. The product was developed for degradation of benzene derivatives, phenols and cresols, naphthalenes, gasolines, kerosene, cyanides (properly diluted), and other toxic wastes from refineries, pesticides, pulp mills, steel mills, and textile and food processing plants (Polybac, 1978). The product offers some advantages over acclimated bacteria taken directly from treatment plants. Starting with acclimated bacteria, Polybac uses radiation to increase the genetic diversity of microorganisms. The strains resulting from radiation processes are further exposed to selected compounds in order to isolate those organisms with enhanced abilities to degrade specific compounds (Zitrides, 1978).

Phenobac or other acclimated cultures may be added to conventional systems to increase process stability, initiate recovery from shock loads, provide faster startup, and increase the capacity to handle variations in organic loads.

Mobay Chemical Corporation recently used Phenobac in order to recover biological activity after a down time period that occurred during extremely cold weather. The activated sludge process, treating mixed organic chemical intermediates, was brought back into operation after 15 days.

Use of Phenobac was also found to enhance biological treatment stability at an Exxon Chemical Company Plant treating petrochemical wastewaters (Polybac, 1978).

3.5.2 Design and Construction Considerations

Phenobac or other acclimated cultures are used as the biomass in conventional or pure oxygen treatment systems, or in landfarming applications. Application rates are determined on a case-by-case basis.

3.5.3 Advantages and Disadvantages

Table B-12 summarizes the advantages and disadvantages of treatment with acclimated cultures.

3.5.4 Costs

Freeze-dried cultures available through Polybac Corp. cost $23.75/lb rehydrated. Fifty-five gallons of water are generally used to rehydrate 25 lbs (Krupka, 1980). All other costs are the same as those encountered for conventional or pure oxygen-activated sludge.

TABLE B-12

ADVANTAGES AND DISADVANTAGES OF TREATMENT WITH ACCLIMATED BACTERIA

Advantages	Disadvantages
● Use of acclimated cultures can decrease start-up time	● May require continuous seeding where there are other microbial predators, excessive washout, or adverse environmental conditions such as presence of toxic metals
● Provides increased resistance to shock loads	
● Increases process stability	● Increased materials costs for operation of activated sludge or high oxygen systems

3.6 ANAEROBIC, AEROBIC, AND FACULTATIVE LAGOONS

3.6.1 General Description and Applications

Lagoons or waste stabilization ponds are large shallow basins that rely on long retention times and natural aeration to decompose the waste. The aerated lagoon is a variation in which the wastes are artificially aerated with diffused air or mechanical aerators. It differs from activated sludge processes in that there is no sludge biomass recycle. Waste stabilization ponds are more sensitive to high concentrations of inorganics and suspended solids than are other biological methods. Since there is no mixing, suspended solids would settle in the pond, creating an excessive load which inhibits benthic microorganisms and creates a sludge blanket along the bottom of the pond.

Waste stabilization ponds have been used to treat low strength industrial wastes or as a polishing step for certain waste types. This treatment module is employed in food processing industries, paper and pulp mills, textile mills, refineries, and petrochemical plants (De Renzo, 1978; Nemerow, 1978).

3.6.2 Design and Construction Considerations

Each subtype of waste stabilization pond utilizes a different type of bacteria but is of similar construction, with an earthen pit and earthen side levees. Treatment of leachates requires that the pond be lined. The designs of various waste stabilization ponds and aerobic lagoons differ significantly. Table B-13 summarizes the major design criteria. The criteria indicate that, in general, lagoons can treat only low strength waste and therefore will be best suited as a polishing step used in conjunction with other treatment methods.

As Table B-13 indicates, the aerobic lagoon requires the greatest surface area to treat an equivalent waste load. Oxygen transfer depends on the ratio of lagoon surface area to volume (length to wide ratio should be less than 3:1), temperature, turbulence, and bacterial oxygen uptake. The system has the least tolerance for high organic loads but benefits from a short detention time (EPA, 1979; Liptak, 1974).

Anaerobic stabilization ponds require significantly less surface area and can handle substantially higher organic loads. Deep lagoons benefit from better heat retention, and an effluent length-to-width ratio of 2:1 is recommended.

Sludge buildup is much less for the anaerobic pond than for the aerobic; for every pound of BOD destroyed by the anaerobic process, about 0.1 lb of solids is formed, as compared to 0.5 lb for the aerobic lagoon. The major disadvantage of the anaerobic lagoon is that it produces strong odors unless the sulfate concentration is maintained below 100 mg/ℓ (Liptak, 1974).

The facultative lagoon benefits from having an aerobic layer that oxidizes hydrogen sulfide gas to eliminate odors. It can handle BOD loads intermittently between the anaerobic and aerobic lagoon.

Artificial aeration with mechanical or diffused aerators allows for deeper basins and higher organic loads than those obtained in aerobic lagoons. The basins are designed for partial mixing only, and anaerobic decomposition occurs on the bottom. Operating costs are significantly less than for activated sludge, but the system cannot withstand the organic loads tolerated by activated sludge.

In general, the use of several lagoons in series is more efficient than one lagoon since it can reduce short-circulation and lead to increased organic removal efficiency.

TABLE B-13

MAJOR DESIGN CRITERIA FOR LAGOONS

	Facultative lagoon	Anaerobic lagoon	Aerobic lagoon	Aerated lagoon
Depth (ft.)	2 - 5	8 - 20	1 - 2	6 - 20
Organic load (lb. BOD/acre/d)	10 - 100	220 - 2000	100 - 200	10 - 300
Detention time (days)	7 - 30	30 - 50	2 - 6	3 - 10
Influent BOD concentration (mg/ℓ)	200 - 500	500 and up	200	200 - 500

Source: U.S. EPA, 1979a; Liptak, 1974

3.6.3 Advantages and Disadvantages

Table B-14 summarizes advantages and disadvantages of waste stabilization ponds and aerated lagoons.

TABLE B-14

ADVANTAGES AND DISADVANTAGES OF STABILIZATION PONDS AND AERATED LAGOONS

Advantages

- Operating costs are low compared to other biological treatment methods
- Cost-effective treatment for polishing effluent
- Waste stabilization ponds require minimal energy

Disadvantages

- Tolerate low strength wastes only
- Intolerant of suspended solids and metals
- Require large land areas
- Performance markedly affected by temperature, and treatment method is not suitable for freezing temperatures
- System has limited flexibility
- Volatile gases may be emitted from processes

3.6.4 Costs

1976 costs for anaerobic and facultative stabilization ponds and aerated lagoons are shown in the following figures based upon the design criteria listed.

Anaerobic Lagoons (Figures B-25 and B-26)

COSTS -
Service Life: 50 years
Average detention time = 35 days; January 1979 dollars; ENR Index = 2872; depth = 10 ft; BOD_5 loading = 466 lb/acre/d. Construction cost includes excavating, grading and other earthwork and service roads. Costs do not include land and pumping. Liner cost not included in estimate.

Wastewater Characteristics:

	Influent, mg/l	Effluent mg./l
BOD_5	600	240

To adjust costs for other BOD_5 and /or detention times, enter curve at effective flow (Q_E)

$$Q_E = Q_{Design} \times \frac{(466 \text{ lb/acre/d})\ (\text{new detention time})}{(\text{new design loading})\ (35 \text{ days})}$$

FIGURE B-25

1976 CONSTRUCTION COST FOR

ANAEROBIC LAGOONS

(Source: EPA, 1978)

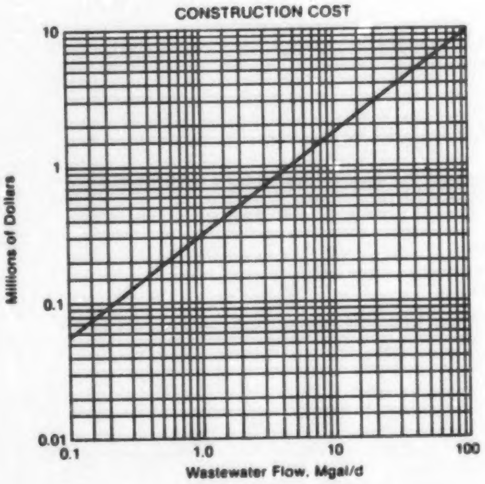
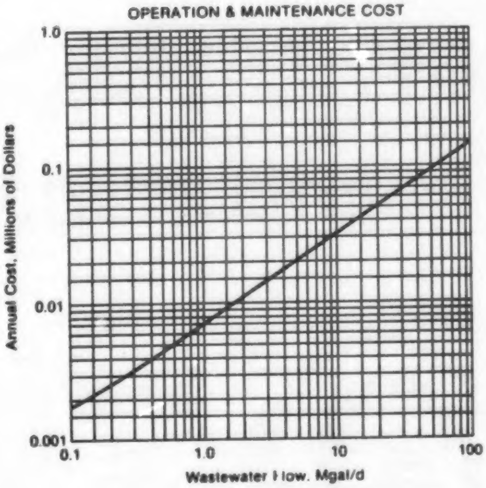


FIGURE B-26

O&M COSTS FOR ANAEROBIC LAGOONS

(Source: EPA, 1978)



Facultative Lagoons (Figures B-27 and B-28)

COSTS-

1. Warm climate - Lagoon loading = 40 lb BOD_5/acre/d.
2. Cool climate (northern U.S.) - Lagoon loading = 20 lb BOD_5/acre/d.
3. Water depth = 4 ft.
4. Construction cost includes excavating, grading, and other earthwork required for normal subgrade preparation and service roads. Costs do not include land and pumping.
5. Process performance:

	Wastewater Characteristics	
	In	Out
BOD_5, mg/ℓ	210	30
COD, mg/ℓ	400	100
TSS, mg/ℓ	230	60
Total-P, mg/ℓ	11	8
NH_3-N, mg/ℓ	20	15 (cool climate) 1 (warm climate)

6. No liner included in cost estimate.
7. ENR Index = 2475

Adjustment Factor: To adjust costs for loadings other than those above, enter curve at effective flow (Q_E).

FIGURE B-27

CONSTRUCTION COSTS FOR FACULTATIVE LAGOONS (1976 COSTS)

(Source: EPA, 1978)

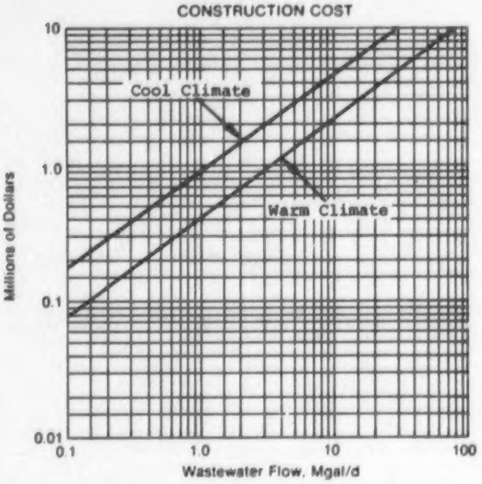
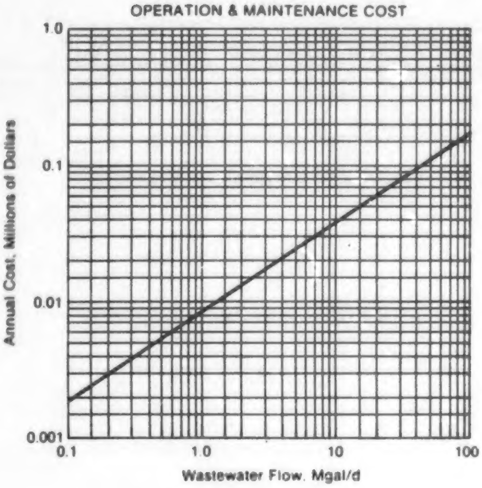


FIGURE B-28

O&M COSTS FOR FACULTATIVE LAGOONS (1976 COSTS)

(Source: EPA, 1978)



Aerated Lagoons (Figures B-29 and B-30)

COSTS -

1. Service Life: 30 years; ENR Index = 2475
2. Theoretical detention time = 7 d; 15-ft water depth; floating mechanical aerators.
3. Horsepower required = 36 hp/Mgal of capacity; power @ $.02/kWh.
4. Construction cost includes excavation, embankment, and seeding of lagoon/slopes (3 cells); service road and fencing; riprap enbankment protection; hydraulic control works; aeration equipment; and electrical equipment.

	Wastewater Characteristics	
	In	Out
BOD_5, mg/ℓ	210	25
COD, mg/ℓ	400	50
TSS, mg/ℓ	230	40
Total-P, mg/ℓ	11	8
NH_3 -N, mg/ℓ	20	18

To adjust construction cost for detention time other than above, enter curve at effective flow (Q_E).

FIGURE B-29

CONSTRUCTION COSTS FOR AERATED LAGOONS

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Detention Time}}{\text{7 days}}$$

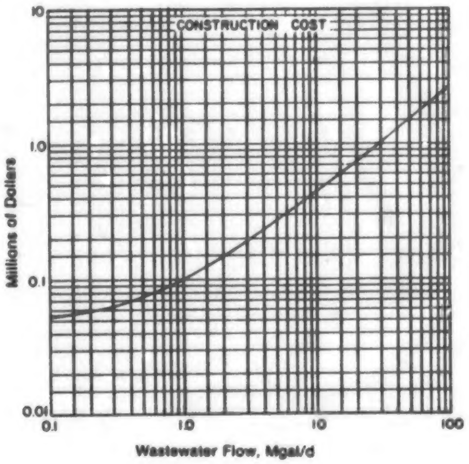
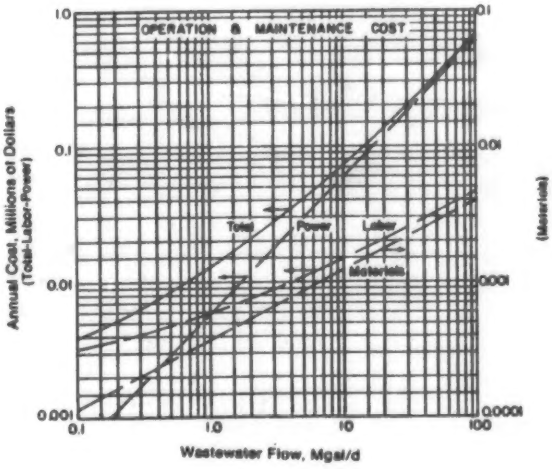


FIGURE B-30

O&M COSTS FOR AERATED LAGOONS



4.0 CARBON ADSORPTION

4.1 GENERAL DESCRIPTION

The process of carbon adsorption involves contacting a waste stream with carbon, which selectively adsorbs hazardous materials by physical and/or chemical forces. When carbon reaches its ultimate capacity for adsorption, that is, when rate of adsorption and desorption are equal, the carbon is removed for disposal, destruction, or regeneration.

4.2 APPLICATIONS

The suitability of carbon adsorption for treatment of wastewater associated with disposal sites depends upon the influent characteristics, the extent of pretreatment. and the required effluent quality. The highest concentration of solute in the influent stream that has been treated on a continuous basis is 10,000 ppm TOC, and a 1 percent solution is currently considered as the upper limit (De Renzo, 1978).

Concentrations of oil and grease in the influent should be limited to 10 ppm. Concentrations of suspended solids should be less than 50 ppm in upflow systems; downflow systems can handle concentrations as high as 2000 ppm, although frequent backwashing would be required. Removal of inorganics by carbon generally requires concentrations of less than 1,000 ppm and preferably less than 500 ppm (De Renzo, 1978).

The suitability of using activated carbon for removal of a certain solute(s) depends upon molecular weight, structure, and solubility. Table B-15 summarizes the influence of molecular structure and other properties of organics on their adsorbability. Table B-16 summarizes the potential for removal of inorganics by activated carbon.

As would be expected from the information in Table B-15, activated carbon has been proven effective in the removal of a variety of chlorinated hydrocarbons, organic phosphorus, carbonates, PCB's, phenols, and benzenes. Specific hazardous organics that are effectively removed include aldrin, dieldrin, endrin, DDD, DDE, DDT, toxaphene, and 2 aroclors (Culp, 1978).

Activated carbon treatment has not been shown to be suitable for treatment of municipal landfill leachates from young landfills; carbon shows poor adsorption capacity for fatty acids, which are prevalent in municipal landfill leachate. Carbon adsorption is generally not effective for wastes with high BOD/COD and COD/TOC ratios (EPA, 1977).

TABLE B-15

EFFECTS OF MOLECULAR STRUCTURE AND OTHER FACTORS ON ADSORPTION BY ACTIVATED CARBON

1. Aromatic compounds are generally more adsorbable than aliphatic compounds of similar molecular size.

2. Branched chains are usually more adsorbable than straight chains.

3. Substituent groups affect adsorbability:

Substituent group	Nature of influence
Hydroxyl	Generally reduces adsorbability; extent of decrease depends on structure of host molecule
Amino	Effect similar to that of hydroxyl but somewhat greater. Many amino acids are not adsorbed to any appreciable extent
Carbonyl	Effect varies according to host molecule; glyoxylic and more adsorbable than acetic but similar increase does not occur when introduced into higher fatty acids
Double bonds	Variable effect
Halogens	Variable effect
Sulfonic	Usually decreases adsorbability
Nitro	Often increases adsorbability

4. An increasing solubility of the solute in the liquid carrier decreases its adsorbability.

5. Generally, strongly ionized solutions are not as adsorbable as weakly ionized ones; i.e., undissociated molecules are, in general, preferentially adsorbed.

6. The amount of hydrolytic adsorption depends on the ability of the hydrolysis to form an adsorbable acid or base.

7. Unless the screening action of the carbon pores intervene, large molecules are more sorbable than small molecules of similar chemical nature. This is attributed to more solute carbon chemical bonds being formed, making desorption more difficult.

Source: Azad 1976
See Copyright Notice, Page 497

TABLE B-16

POTENTIAL FOR REMOVAL OF INORGANIC MATERIAL BY ACTIVATED CARBON

Constituents	Potential for removal by carbon
Metals of high sorption potential	
Antimony	Highly sorbable in some solutions
Arsenic	Good in higher oxidation states
Bismuth	Very good
Chromium	Good, easily reduced
Tin	Proven very high
Metals of good sorption potential	
Silver	Reduced on carbon surface
Mercury	CH_3HgCl sorbs easily Metal filtered out
Cobalt	Trace quantities readily sorbed, possibly as complex ions
Zirconium	Good at low pH
Elements of fair-to-good sorption potential	
Lead	Good
Nickel	Fair
Titanium	Good
Vanadium	Variable
Iron	FE^{3+} good, FE^{2+} poor, but may oxidize
Elements of low or unknown sorption potential	
Copper	Slight, possibly good if complexed
Cadmium	Slight
Zinc	Slight
Beryllium	Unknown
Barium	Very low
Selenium	Slight
Molybdenum	Slight at pH 6-8, good as complex ion
Manganese	Not likely, except as MnO_4
Tungsten	Slight

--continued--

TABLE B-16 (Continued)

Constituents	Potential for removal by carbon
Miscellaneous inorganic water constituents	
Phosphorus	
P, free element	Not likely to exist in reduced form in water
PO_4^{3-} phosphate	Not sorbed but carbon may induce precipitation $Ca_3(PO_4)_2$ or
Free halogens	
F_2 fluorine	Will not exist in water
Cl_2 chlorine	Sorbed well and reduced
Br_2 bromine	Sorbed strongly and reduced
I_2 iodine	Sorbed very strongly, stable
Halides	
F^- flouride	May sorb under special conditions
Cl^-, Br^-, I^-	Not appreciably sorbed

Source: Culp, 1978
See Copyright Notice, Page 497.

4.3 DESIGN AND CONSTRUCTION CONSIDERATIONS

Critical design criteria are organic load, hydraulic load, contacting method, contact time, and regeneration requirements.

The approximate carbon requirements for a specific organic load, and the residual organic levels can be estimated from adsorption removal kinetics conducted on a batch basis. An isotherm can be used as a functional expression for variation of adsorption with concentration of adsorbate in bulk solution. The isotherm is expressed in terms of removal of impurity (i.e., BOD, COD, or color).

$$\frac{X}{M} = KC\ 1/n$$

where: X = impurity adsorbed
M = weight of carbon
C = equilibrium concentration of impurity
K,n = constant (Culp, 1978)

Isotherms are a useful approximation of treatability, but generally give a falsely high estimate of continuous carbon performance.

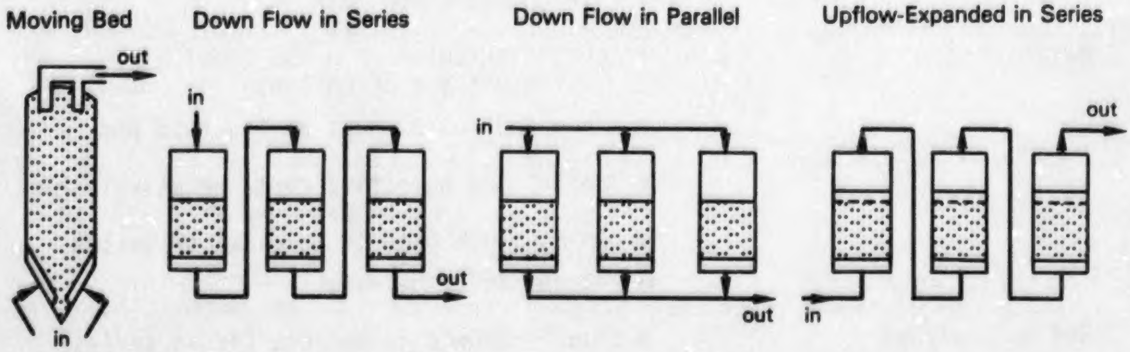
There are four basic ways that waste streams can be contacted, and the choice of the appropriate method depends upon influent characteristics, effluent criteria, flow rate, and economics. Table B-17 summarizes these available methods and Figure B-31 illustrates them.

Upflow beds have the advantage over downflow beds in that they more closely approach continuous countercurrent contact operations, which results in minimal use of carbon. Also, upflow beds can be designed to allow for removal of spent carbon and addition of fresh carbon while the columns remain in operation. Downflow columns have the advantage that they can handle higher suspended solids concentrations, although backwashing requirements may be frequent.

FIGURE B-31

ADSORBER CONFIGURATIONS

(Source: DeRenzo, 1978)



Typical operating parameters for carbon adsorption systems are summarized in Table B-18. The parameters are based on system operations for physical/ chemical and tertiary treatment systems.

TABLE B-17

SUMMARY OF ACTIVATED CARBON CONTACTING METHODS

Method	Comments
Downflow adsorbers in parallel	● For high volume applications ● Can handle higher than average suspended solids (∿65-70 ppm) ● Relatively low capital costs ● Effluents from several columns blended, therefore less suitable where effluent limitations are low ● 2-10 gpm/ft^2 flow rate
Downflow adsorbers in series	● Large volume systems ● Countercurrent carbon use ● Effluent concentrations relatively low ● Can handle higher than average suspended solids (∿65-70 ppm) if downflow ● Capital costs higher than for parallel systems ● 2-10 gpm/ft^2 flow rate
Moving bed	● Countercurrent carbon use (most efficient use of carbon) ● Suspended solids must be low (<10 ppm) ● Capital and operating costs relatively high ● Can use such beds in parallel or series ● 2-7 gpm/ft^2 flow rate
Upflow-expanded	● Countercurrent carbon use (if in series) ● Can handle high suspended solids (they are allowed to pass through) ● High flows in bed (∿15 gpm/ft^2) ● Minimum pretreatment ● Minimum headloss

Source: DeRenzo, 1978

TABLE B-18

OPERATING PARAMETERS FOR CARBON ADSORPTION

Parameter	Requirements	Sources
Contact time	Generally 10-50 min; may be as high as 2 hours for some industrial wastes	Culp, 1978 Liptak, 1974
Hydraulic load	2-15 gpm/ft^2 depending on type of contact system; see Table B-17	---
Backwash rate	Rates of 20-30 gpm/ft^2 usually produce 25-50 percent bed expansion	Liptak, 1974
Carbon loss during regeneration	4-9 percent	DeRenzo, 1978
	2-10 percent	Liptak, 1974
Weight of COD Removed per weight of carbon	0.2 - 0.8	Culp, 1978 DeRenzo, 1978
Carbon requirements		
PCT plant	500 - 1800 lb/10^6 gal.	Culp, 1978
Tertiary plant	200 - 500 lb/10^6 gal.	
Bed depth	10 - 30 feet	EPA, 1978a

The decision to regenerate and reuse granular carbon or to use it on a once-through basis is based primarily on economics. For plants requiring less than 200 lb/day of carbon (less than approximately 0.8 mgd), regeneration is probably not economical. Most leachate treatment facilities will fall within this range.

Use of electric furnaces, rather than multiple hearth furnaces, may make it possible to regenerate activated carbon economically for plants using less than 200 lb/day (Culp, 1978). Regeneration needs can be determined on the basis of COD adsorbed per pound of carbon or required carbon dosage in terms of total flow.

4.4 ADVANTAGES AND DISADVANTAGES

Advantages and disadvantages of carbon adsorption are summarized in Table B-19.

TABLE B-19

ADVANTAGES AND DISADVANTAGES OF ACTIVATED CARBON

Advantages	Disadvantages
● High flexibility in operation and design	● Intolerant of high suspended solids levels
● Suitable for treatment of a wide range of organics that do not respond to biological treatment	● Requires pretreatment for oil and grease removal where concentrations are greater than 10 ppm
● Has high adsorption potential for some highly hazardous inorganics (i.e., CR, CN)	● Not suitable for removal of low molecular weight organics, highly soluble or highly ionized organics
● Tolerant of some fluctuations in concentrations and flow	● Limited in practice to wastes with less than 10,000 ppm organics
	● O&M costs are high

4.5 COSTS

Costs for 1976 are shown in Figures B-32 and B-33 for tertiary granular activated carbon adsorption. The costs are based on the following criteria:

COSTS - ENR Index = 2475

1. Construction cost includes vessels, media, pumps, carbon storage tanks, controls, and operations building; loading rate = 30 lb/Mgal; contact time = 30 min; disposal costs not included.

2. O&M cost includes pumping ($0.02/kWh), labor ($7.50/h, including fringes) and maintenance.

3. No regeneration is included; therefore, above 3 Mgal/d, cost curves are extrapolated.

FIGURE B-32

CONSTRUCTION COSTS FOR TERTIARY ACTIVATED CARBON TREATMENT

(1976 COSTS)

(Source: EPA, 1978)

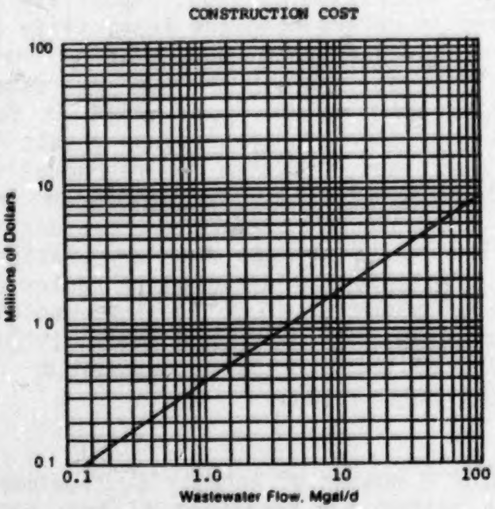
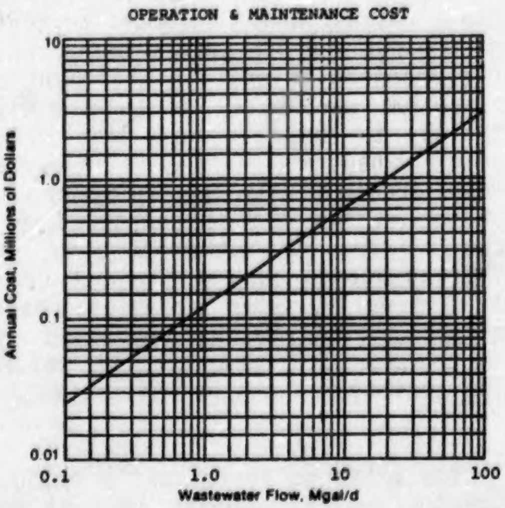


FIGURE B-33

O&M COSTS FOR TERTIARY ACTIVATED CARBON TREATMENT

(1976 COSTS)

(Source: EPA, 1978)



5.0 ION EXCHANGE

5.1 GENERAL DESCRIPTION

Ion exchange resins are insoluble solids containing fixed cations and anions capable of reversible exchange with mobile ions of the opposite sign in solutions with which they are brought into contact. The direction and extent of the reaction are governed by the relative insolubilities of the salts that can be formed and the equilibrium constants (De Renzo 1978).

5.2 APPLICATIONS

Ion exchange is considered applicable for removal of the following classes of chemicals:

- All soluble metallic elements, either cationic or anionic
- Anions such as halides, cyanides, and nitrate
- Acids such as carboxylics, sulfonics, and some phenols at pH sufficiently alkaline to give the ions

However, there are certain limitations on the ion exchange capability of various resins, and these must be considered in determining the feasibility of ion exchange and its pretreatment requirements. The upper concentration limit for exchangeable ions for efficient operations is about 2,500 mg/ℓ expressed as calcium carbonate, or 0.05 equivalents/liter. This upper limit is due primarily to the fact that high concentrations of exchangeable ions result in a rapid exhaustion of the resin during the process and costs for regeneration become prohibitively high (DeRenzo, 1978). Also, the effectiveness of ion exchange resins can be decreased by the presence of certain waste constituents. Suspended matter must be very low so as not to foul the resins. Oxidizing agents such as chromic or nitric acid can be damaging to resins as well. Finally, some organics, especially aromatics, can be irreversibly adsorbed by the resin, resulting in decreased capacity. (DeRenzo, 1978). This problem can sometimes be solved by prefiltering the wastewater or by using scavenger exchange resins (Metcalf and Eddy, 1972).

Ion exchange is currently being used in a number of industrial treatment processes, which suggests that it may be suited for treatment of some hazardous waste leachates. Notably, ion exchange is widely used in the electroplating industry to remove impurities from rinse water. Rinse waters are usually fairly dilute solutions of chromium, nickel, and cyanides. Ion exchange is generally used as a polishing step in treatment of electroplating wastes and is also widely used as a final treatment method for metal finishing wastewaters for removal of cyanides, zinc, chromium, and other metals.

Another application is for removal of valuable metals such as copper, molybdenum, cobalt, and nickel from dilute leach liquors from tailing or dump piles (Nemerow, 1978; De Renzo, 1978).

5.3 DESIGN AND CONSTRUCTION CONSIDERATIONS

The major design considerations for ion exchange treatment include selection of the appropriate resin, based on organic loads, hydraulic load, selection of the appropriate operating mode, and consideration of backwashing and regeneration requirements.

5.3.1 Selection of the Resin

The extent to which removal of anions and/or cations occurs depends on the equilibrium that is established between the ions in the aqueous phase and those in the solid phase. For instance, the equilibrium for the removal of sodium from solution is defined as follows:

$$\frac{[H] \times RNa}{[Na] \times RH} = KH \rightarrow Na$$

where: $KH \rightarrow Na$ = selectivity coefficient
$x\ RH$ = mole fraction of hydrogen on the exchange resin
$x\ RNa$ = mole fraction of sodium on exchange resin
[ ] = concentration in the solution phase

The selectivity coefficient depends on the nature and volume of the ion, the type of resin and its saturation and the ion concentration in wastewater (Metcalf and Eddy, 1972). Since the stability of the salts formed by the ions and exchangers can be highly variable, it is important to make a knowledgeable choice of the exchange material so that it will allow for the selective separation. Exchange resins can be selected and compared by the following criteria:

- Functionality, which refers to the kinds of ions that are exchanged
- Exchange capacity, which is a measure of the total uptake of specific ions
- Selectivity, which refers to the preference of one kind of exchangeable ion over another (De Renzo, 1978)

Table B-20 lists some available resin types that may be well-suited to leachate treatment.

TABLE B-20

COMMON REACTIVE GROUPS FOR ION EXCHANGE RESIN[1]

Reactive group	Exchangeable ions
Strong acid (sulfonic)	Cations in general
Weak acid (carboxylic)	Cations in general
Weak acid (phenolic)	Cesium and polyvalent cations
Strong base (quaternary amine)	All anions, especially used for anions of weak acids (cyanide, caronate, silicate, etc.)
Weak base (tertiary and secondary amine)	Anions of strong acids (sulfate, chloride, etc.)
Chelating (varied, may be iminodiacetate or oximine groups)	Cations, especially transition and heavy elements

---

[1] Differences in the particular starting materials and preparation route frequently give rise to differences in handling properties, stability and reaction kinetics between resins that have the same polymer backbone, functional groups, and exchange capacity. Hence it is important to test a variety of resins for a particular application.

Source: DeRenzo, 1978

5.3.2 Mode of Operation

Ion exchange may be carried out as a batch or continuous operation. Where a continuous operation is practiced, there are three possible operating modes: cocurrent fixed bed, countercurrent fixed bed, and countercurrent continuous. Table B-21 presents a summary comparison of the three processes (De Renzo, 1978).

Commonly used variations of the fixed bed exchange mode include mixed beds and use of exchange columns in series. Mixed beds (Figure B-34) give a larger driving force and yield higher removal of efficiencies than the same amount of resin used in separate beds (Vermuelen, 1977). Where a number of beds are used in series (Figure B-35), it is possible to detach the upstream bed, regenerate it, and reattach it at the downstream end, thereby making it more similar to a countercurrent stream.

5.3.3 Regeneration

Continued contact of the exchange resin with the solution containing the ions to be removed results in eventual exhaustion of the active sites on the resins. It is generally advisable to regenerate the resins before all active sites are exhausted. Table B-22 summarizes the types of regenerants and dosage ranges. Optimum regenerant quantities and conditions will vary with the process involved and can be determined experimentally.

TABLE B-21

COMPARISON OF ION EXCHANGE OPERATING MODES

	Cocurrent fixed bed	Countercurrent fixed bed	Countercurrent continuous
Capacity for high feed flow & Conc.	Least	Middle	Highest
Effluent quality	Fluctuates with bed exhaustion	High, minor fluctuations	High
Regenerant and rinse requirements	Highest	Somewhat less than cocurrent	Least, yields most conc. regenerant waste
Equipment complexity	Simplest; can use manual operation	More complex; automatic controls for regeneration	Most complex; completely automated
Equipment for continous operation	Multiple beds, single regeneration equipment	Multiple beds, single regeneration equipment	Provides continous service
Relative costs (per unit volume)			
Investment	Least	Middle	Highest
Operating	Highest chemicals & labor; highest resin inventory	Less chemicals, water & labor than concurrent	Least chemical & labor; lowest resin inventory

Source: DeRenzo, 1978

FIGURE B-34

MIXED BED OPERATING CYCLE SHOWING (A) SERVICE PERIOD,
(B) BACKWASH WITH RESIN SEGREGATION, (C) REGENERATION, AND (D) RESIN RE-MIXING

(Source: Vermuelen, 1977)
See Copyright Notice, Page 497

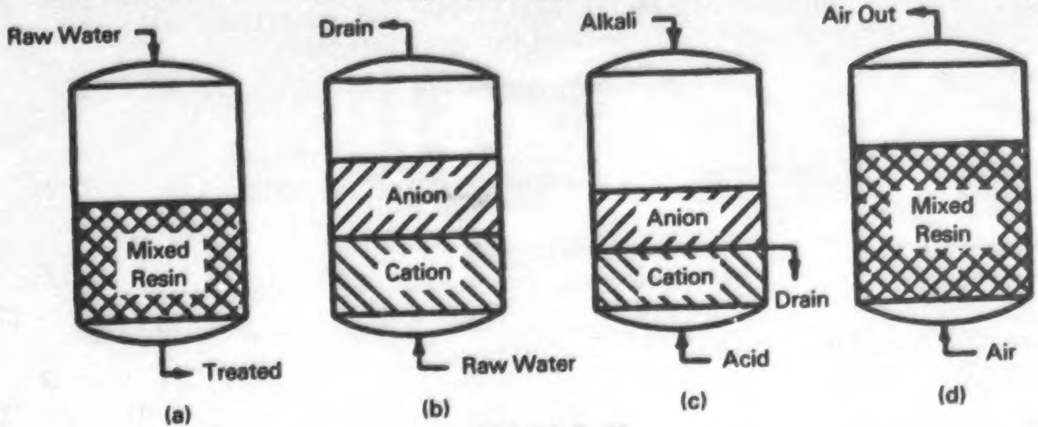

Raw Water
Mixed Resin
Treated
(a)
Drain
Anion
Cation
Raw Water
(b)
Alkali
Anion
Cation
Drain
Acid
(c)
Air Out
Mixed Resin
Air
(d)

FIGURE B-35

CYCLIC MULTIBED SYSTEM, EACH STEP HAVING TWO BEDS IN PROCESS USE

(1 AHEAD OF 2) AND ONE BED OFFSTREAM FOR REGENERATION

(Source: Vermuelen, 1977)
See Copyright Notice, Page 497

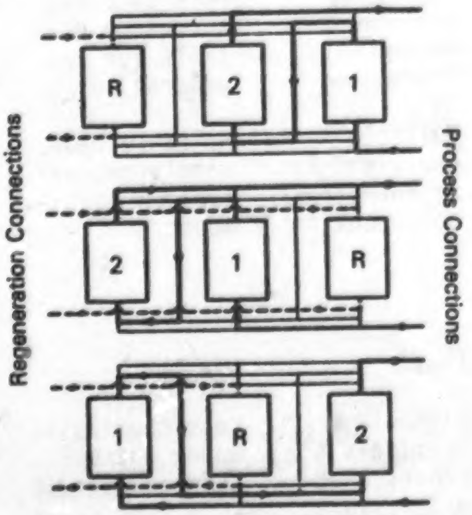

Regeneration Connections
Process Connections
R
2
1
2
1
R
1
R
2

TABLE B-22

EXAMPLES OF REGENERANTS AND DOSAGE RANGES

Resin type	Desired Ionic form	Regenerant	lb/ft^3	Concentration
Cationic	H^+	HCl	4-10	2-10%
	H^+	H_2SO_4	6-12	2-10%
	Na^+	NaCl	5-10	6-25%
Strong basic anionic	OH^-	NaoH	4-8	2-10%
Weakly basic anionic	NH_3	Free base	1-2	2-4%
	NaOH	Free base	2-4	1-2%

5.4 ADVANTAGES AND DISADVANTAGES

Advantages and disadvantages of ion exchange for treatment of leachate are summarized in Table B-23.

5.5 COSTS

Costs of ion exchange vary widely, depending upon stream size and concentration of contaminants. Treatment costs have been estimated to be on the order of $\$6/10^3$ gal. for dilute, complex waste streams (DeRenzo, 1978).

To give the reader an idea of the costs associated with ion exchange, two examples have been used. Example 1 presents 1978 costs associated with ion exchange removal of nitrate, sulfate, and other anions for a range of flows from 70,000 gpd to 830,000 gpd. Construction, operation, and maintenance costs are shown in Tables B-24 and B-25 for the following conditions:

- Nitrate and sulfate concentrations of 100 mg/ℓ and 80 mg/ℓ respectively; other anions, 120 mg/ℓ
- Strongly basic anionic exchange resin

TABLE B-23

ADVANTAGES AND DISADVANTAGES OF ION EXCHANGE

Advantages	Disadvantages
● Suitable for removal of soluble inorganics not removed by precipitation/sedimentation	● Not suitable for removal of high concentrations
● Technology is reasonably well demonstrated for electroplating wastes, metal, and pickling liquors	● Pretreatment is required for suspended solids, certain organics (especially aromatic), and oxidants
● Process energy requirements are low	● Operation and maintenance costs are high compared to most treatment processes
	● Spent regenerant has potential for containing high concentrations of contaminants

---

- Bed-depth of 6 feet
- Regenerate requirements: 15 lb resin/ft^3
- Contact vessel of prefabricated steel with the conceptual designs shown in Table B-24.
- Regeneration time and backwash time of 54 min. and 10 min., respectively
- Costs for spent regenerant excluded

Costs for treatment of a complex, dilute waste containing heavy metal cations may differ significantly from the costs presented above for treatment of a dilute waste containing anions. The major variables that may affect the costs include the number of contactors needed, the types of resins required, and regeneration requirements. Strongly basic anion exchange resins used to cost the treatment system described above tend to be the most expensive. Cost ranges for the various types of resins are summarized below:

Strongly basic anion exchangers: \$140-280/ft^3
Strongly acid cation exchangers: \$ 50- 70/ft^3
Weakly basic anion exchangers: \$130-170/ft^3
Weakly acid cation exchangers: \$100-120/ft^3
(Rohm and Haas, 1979)

Unit costs for various regenerants are as follows:

HCL - \$ 38/ton
H_2SO_4 - \$ 39-66/ton
NaCL - \$ 67/ton
NaOH - \$200/ton
NH_3 - \$155-195/ton

Source: Chemical Market Reporter, 1980

The second example was costed by Arthur D. Little from 1976 cost data. The stream is a dilute mixed waste stream from a metal finishing operation.

Fixed Stream to Process: dilute mixed aqueous waste from metal finishing, 80,000 gallons/day (303m^3/day) containing:

zinc	15 mg/ℓ
copper	0.5 mg/ℓ
cyanide (total)	19 mg/ℓ
chromium	22 mg/ℓ
pH	10 (approximately)

TABLE B-24

CONCEPTUAL DESIGN FOR PRESSURE ION EXCHANGE NITRATE REMOVAL

Plant capacity (gpd)	Number of contactors	Diameter of contactors (ft)	Housing ft^2
70,000	2	2	132
270,000	2	4	210
425,000	2	5	255
610,000	2	6	304
830,000	2	7	357

(Source: EPA, 1979)

TABLE B-25

1978 CONSTRUCTION COST FOR PRESSURE ION EXCHANGE NITRATE REMOVAL

	Plant capacity (gpd)				
Cost category	70,000	270,000	452,000	610,000	830,000
Excavation and sitework:	$50	$110	$140	$170	$200
Manufactured equipment					
Equipment	11,860	16,500	19,090	21,660	25,920
Media	5,460	21,860	34,160	49,200	66,960
Concrete	280	490	550	740	880
Steel	420	680	950	1,110	1,300
Labor	4,770	5,990	6,880	7,590	9,250
Pipe and valves	9,650	12,440	13,600	15,360	18,350
Electrical and instrumentation	18,390	21,460	23,070	23,720	24,360
Housing	7,600	8,900	9,800	10,700	11,600
SUBTOTAL	58,480	88,430	108,240	130,250	158,820
Misc. and contingency	8,770	13,260	16,240	19,540	23,820
TOTAL	67,250	101,690	124,480	149,790	182,640

Source: EPA, 1979

Treatment Objective: to provide effluent that meets discharge requirements for zinc (</mg/ℓ), copper, chromium and cyanide (all <0.3 mg/ℓ), pH 6-8.5; cyanide to be separated from metals.

Design Basis: 24-hour/day - 350 days/year
3-bed system, in duplicate for regeneration
24-hour loading time

Product Streams: clean water, metal cations, chromium, cyanide.

TABLE B-26

OPERATION AND MAINTENANCE SUMMARY FOR PRESSURE ION EXCHANGE NITRATE REMOVAL

Plant flow rate (gpd)	Electrical energy (kw-hr/yr)			Maintenance material ($/yr)	Labor (hr/yr)	Total cost[1] ($/yr)
	Building	Process	Total			
70,000	13,540	126	13,666	$ 1,890	1,000	$12,300
270,000	21,550	510	22,060	6,340	1,400	21,000
425,000	26,160	790	26,950	9,660	1,550	25,970
610,000	31,190	1,140	32,330	13,710	1,700	31,680
830,000	36,630	1,550	38,180	18,520	1,900	38,670

[1] Calculated using $.03/kw-hr and $10.00/hr of labor

Source: EPA, 1979

TABLE B-27

ION EXCHANGE COST ESTIMATE (1976 DOLLARS)

	Annual quantity	Cost per unit quantity	Annual cost
Variable costs			
Operating labor	2,200 MM	\$12/hour	\$26,400
Chemicals			
Resin replacement (20%/yr)			2,800
- NaOH (70%)	175 tons	\$60/ton	28,000
- H_2SO_4 (98%)	48 tons	\$53.25/ton	2,600
Total chemicals			\$33,000
Utilities (electricity)	75,000 kWh	\$0.02/kWh	1,500
Maintenance (3% of investment)			12,000
Total variable costs			\$73,300
Fixed costs			
Taxes & insurance (2% of investment)[1]			\$ 8,000
Capital recovery (10 years @ 10%)[1]			64,700
Total fixed costs			\$72,700
Unit costs (\$/10^3 gallons)			\$ 5.21

---

[1] Estimated Capital Investment \$400.00
Source: DeRenzo, 1978

6.0 LIQUID ION EXCHANGE

6.1 GENERAL DESCRIPTION AND APPLICATIONS

Liquid Ion Exchange (LIE) involves the selective removal and/or separation of free and complexed metal ions and other inorganics from aqueous streams. In this process, the inorganic(s) of interest are transferred from the aqueous phase to an immiscible organic phase. This organic phase is then contacted with a second aqueous phase whose composition is such that the inorganics now transfer to that phase. The basic principle underlying the liquid ion exchange reaction is the concept of distribution or partition between phases. In the liquid ion exchange process, a water-soluble, ionic species is caused to become more soluble in an organic solvent (by salt formation, complexation, etc.), thus promoting the partition or extraction of that species into the solvent (DeRenzo, 1978).

LIE is competitive with conventional ion exchange and can be used to treat much higher concentrations than the conventional process. The process is applicable to any aqueous waste stream containing extractable species and to any wastes containing inorganics that can be dissolved in aqueous acid on alkali to yield extractable species. Virtually all soluble cations can be removed. although commercially available extractants preferentially extract heavy polyvalent metals. Soluble but undissociated species such as mercuric chloride, anions, and metal oxyanions, and weak acids such as hydrofluoric acid can also be extracted. (DeRenzo 1978). The process is sensitive to certain wastewater contaminants. The presence of surfactants causes changes in phase separation. Oxidants tend to cause degradation of functional groups of extractants, and the presence of suspended matter in excess of 0.1 percent may have detrimental effects on the process.

Although in theory there is no limit to the concentrations that can be treated by this process, the volume of extractants that must be used places practical limitations on the concentrations. Commercial processes for extraction of cobalt and nickel treat solutions up to 10 g/ℓ and this is probably a typical upperlimit concentration (DeRenzo, 1978).

There are several commercial and near-commercial applications for removal of various inorganics by LIE. Some examples include:

- Recovery of nitric, hydrofluoric, and molybdic acid from metal pickling liquors
- Recovery of copper from spent alkaline etchant solutions and from ammonia/ammonium carbonate leaching of metallic copper scrap
- Recovery of iron, zinc, copper, nickel, and chromium from alkali hydroxide sludges
- Removal of cyanide and zinc from electroplating rinse water (DeRenzo, 1978)

6.2 DESIGN AND CONSTRUCTION CONSIDERATIONS

Liquid ion exchange is a steady state process because of its dependence on a constant distribution coefficient and on proper time for phase separation. The contacting process should provide intimate mixing to maximize mass transfer. Therefore, the process should be run on a continuous basis rather than as a batch operation.

Three types of contactors are currently available: mixed settlers, differential contactors, and centrifugal contactors. Mixed settlers are the simplest and most commonly used, but they are least flexible in that they are least able to respond to changes in process conditions such as feed composition. These units require a long time for contact and phase separation. Centrifugal contactors have a very short detention time, and provide excellent phase separation. They are able to respond quickly to process changes but are complex and require an intricate control system (DeRenzo, 1978).

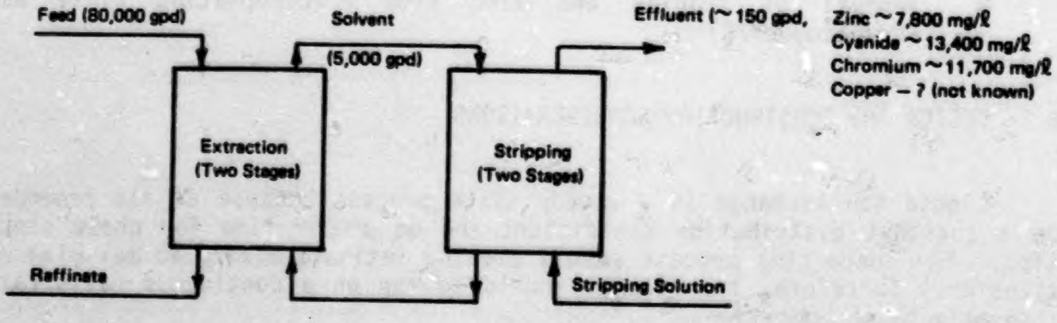
Extraction reagents are classified according to differences in the nature of stripping chemistry. The reagents are used as dilute solutions (5-30 percent). Classes of reactants include:

- Basic extractants such as ketones, ethers, and amines react with acids or metallic ions to form salts or complexes soluble in organic solvents
- Acidic extractants, such as carboxylic acids, and naphthalenes and alkylnaphalene sulfonic acids react with bases or salts by exchange of cations.
- Chelating extractants for stable chelate complexes with metal ions
- Ionic extractants form organic extractable ion pairs with anions or cations (DeRenzo 1978)

The process yields two aqueous streams, the cleaned stream and the second aqueous stream termed the stripping liquor. Both will contain small amounts of the extraction solvent. The "cleaned" aqueous stream may require further treatment by adsorption prior to stream discharge. The stripping liquor will contain hazardous wastes at high concentrations (see Figure B-36) and must be treated to render the hazardous compounds innocuous (DeRenzo 1978).

FIGURE B-36

LIQUID ION EXCHANGE OF METAL FINISHING WASTES



6.3 ADVANTAGES AND DISADVANTAGES

Advantages and disadvantages of liquid ion exchange are summarized in Table B-28.

TABLE B-28

ADVANTAGES AND DISADVANTAGES OF LIQUID ION EXCHANGE

Advantages	Disadvantages
● Applicable for treatment of most dissolved ionized and un-ionized inorganics in aqueous streams	● Process is sensitive to the presence of oxidants, surfactants, and suspended solids
● Process has been proven reliable in treatment of pickling liquor and electroplating wastes	● There is a potential for water pollution unless reclaimed extractants are stripped from discharge stream
● Process can treat higher inorganic concentrations than conventional ion exchange	● The regeneration solution into which hazardous components are stripped from the extraction solvent will contain hazardous components at high concentrations. These must be rendered innocuous

6.4 COSTS

Few economic studies have been done on the treatment of dilute waste streams. Costs appear to be comparable or somewhat less than conventional ion exchange. A rough idea of costs can be determined from the example presented in Table B-29.

TABLE B-29

LIQUID ION EXCHANGE COST EXAMPLE OPERATING COST ESTIMATES FOR LIQUID ION EXCHANGE (1976 COSTS)

	Annual quantity	Cost per unit quantity	Annual cost
Variable costs			
Operating Labor	2,200 MM	$12/hour	$26,400
Electricity	80,000 kWh	0.02/kWh	1,600
Chemicals			
Kerosene	330/gal	0.45/gal	150
Triaecyl alcohol	700/lbs	0.37/lb	260
Aliquat	90/lbs	1.40/lb	130
Caustic (70%)	70/tons	180/ton	12,600
Total chemicals			$33,000
Maintenance (3% of investment)[1]			12,000
Total variable costs			$53,140
Fixed costs			
Taxes & insurance (2% of investment)[1]			$ 6,000
Capital recovery (10 years @ 10%)			48,860
Total fixed costs			$108,000
Unit costs ($/10 gallons)			$ 4.09

---

[1] Estimated capital investment $300.00.
Source: DeRenzo, 1976

This cost estimate does not include further treatment of the "cleaned" stream to remove any extractant solvent.

7.0 AMMONIA STRIPPING

7.1 GENERAL DESCRIPTION

Ammonia stripping is a mass transfer operation which, at high pH, can decrease ammonia concentrations in water by bringing it into intimate contact with air. At a pH of about 12, only ammonia gas will be present, and ammonia is readily stripped.

7.2 APPLICATION

Ammonia stripping will be desirable for treatment of any leachate where ammonia is present in sufficiently high concentrations to exhibit toxicity to biomass in biological treatment or to create such environmental problems as toxicity to fish or high oxygen demand following steam discharge.

Leachate from municipal refuse can exhibit widely varying ammonia concentrations. A range of 2 mg/ℓ to about 1000 mg/ℓ was reported for leachate from 13 municipal fills (EPA, 1977).

7.3 DESIGN AND CONSTRUCTION CONSIDERATIONS

Ammonia stripping can be carried out either in a stripping lagoon or in a packed column. Schematics of an ammonia stripping lagoon and packed tower are illustrated in Figures B-37 and B-38. The major factors affecting performance and design include pH, temperature, air flow, hydraulic loading, and tower packing depth and spacing (Culp 1978). Cost and performance are relatively independent of influent ammonia concentrations (Culp, 1978).

The pH must be raised to a point where all or nearly all ammonia is converted to gas. The pH for efficient operations ranges varies from about 10.8-11.5. Where lime precipitation is part of a treatment scheme, it is advantageous to locate the ammonia stripping unit after lime precipitation to take advantage of the high pH in the clarifier effluent.

As water temperature decreases, it becomes more difficult to remove ammonia by stripping. The amount of air per gallon must be increased to maintain removal as temperature decreases. It is impractical to heat stripping units when the temperature reaches freezing (Culp, 1978).

The hydraulic loading rate in a packed tower is a critical factor in determining performance. If hydraulic loading becomes too high, good droplet formation needed for efficient stripping is disrupted. If the rate is too low, packing may not be properly wetted, resulting in poor performance and formation of scale. Optimum hydraulic load for a packed tower is about 1 to 2 gpm/ft^2. Optimum air flow rates for packed towers have been shown to be 300-500 ft^3/gal. for 90-95 percent removal.

Where ammonia concentrations are high (in excess of 100 mg/ℓ), it may be attractive both economically and environmentally to recover the ammonia in an adsorption tower. With good countercurrent contact, 90-95 percent of the ammonia can be transferred to the absorption solution. Figure B-39 illustrates the ammonia removal and recovery process.

FIGURE B-37

AMMONIA STRIPPING LAGOON

(Source: Culp, 1978)
See Copyright Notice, Page 497

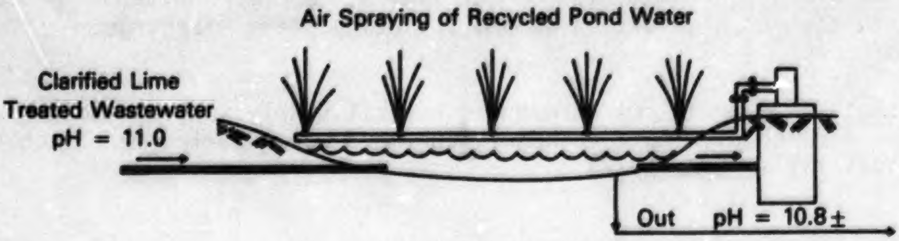
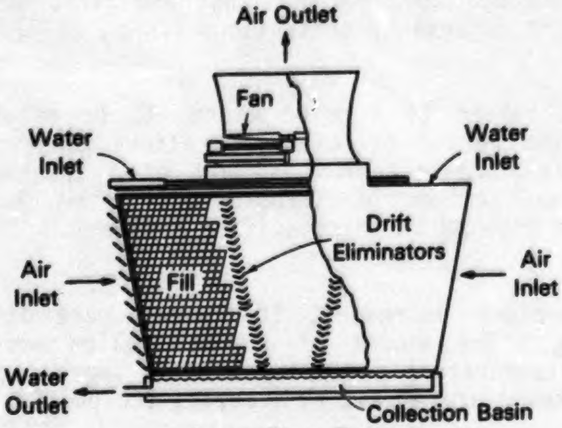

Air Spraying of Recycled Pond Water
Clarified Lime
Treated Wastewater
pH = 11.0
Out
pH = 10.8 ±

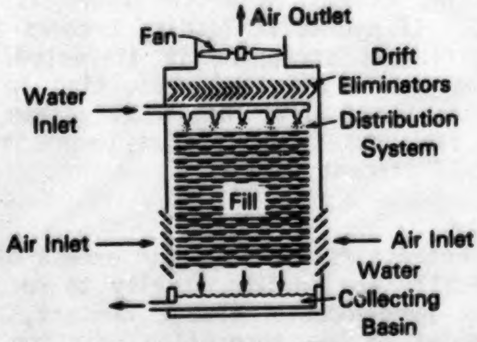
FIGURE B-38

AMMONIA STRIPPING TOWER

(Source: Culp, 1978)
See Copyright Notice, Page 497


Air Outlet
Fan
Water
Inlet
Water
Inlet
Drift
Eliminators
Air
Inlet
Fill
Air
Inlet
Water
Outlet
Collection Basin

Cross-Flow Tower

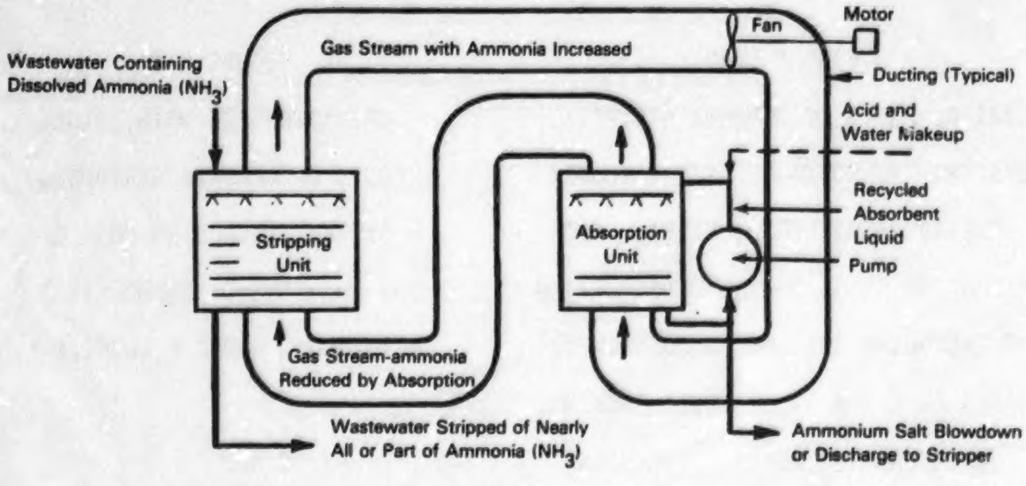

Air Outlet
Fan
Drift
Eliminators
Water
Inlet
Distribution
System
Fill
Air Inlet
Air Inlet
Water
Collecting
Basin

Countercurrent Tower

FIGURE B-39

SCHEMATIC OF AMMONIA REMOVAL AND RECOVERY SYSTEM

(Source: U.S. EPA, 1978)


Motor
Fan
Gas Stream with Ammonia Increased
Wastewater Containing Dissolved Ammonia (NH_3)
Ducting (Typical)
Acid and Water Makeup
Recycled Absorbent Liquid
Stripping Unit
Absorption Unit
Pump
Gas Stream-ammonia Reduced by Absorption
Wastewater Stripped of Nearly All or Part of Ammonia (NH_3)
Ammonium Salt Blowdown or Discharge to Stripper

7.4 ADVANTAGES AND DISADVANTAGES

Table B-30 summarizes the advantages and disadvantages of ammonia stripping.

TABLE B-30

ADVANTAGES AND DISADVANTAGES OF AMMONIA STRIPPING

Advantages	Disadvantages
● Can reduce ammonia levels below toxic level to biomass in biological treatment	● Cost prohibitive to operation at temperatures below freezing
● Process is relatively independent of ammonia concentration	● Sensitive to pH, temperature, and to fluxes in hydraulic load
	● Releases ammonia to air unless recovery is implemented

7.5 COSTS

Figures B-40 and B-41 summarize the capital, operating, and maintenance costs for ammonia stripping towers designed to treat flows ranging from 0.01 - 10 mgd.

FIGURE B-40

CAPITAL COSTS OF AMMONIA STRIPPING SYSTEM, INCLUDING RELATED YARDWORK ENGINEERING, LEGAL, FISCAL, AND FINANCING COSTS DURING CONSTRUCTION AND EXCLUDING COST OF pH ADJUSTMENT. BASED ON 1 gpm/sf OF TOWER PACKING, 24 ft PACKING DEPTH. INCLUDES INFLUENT PUMPING (TDH = 50 ft).

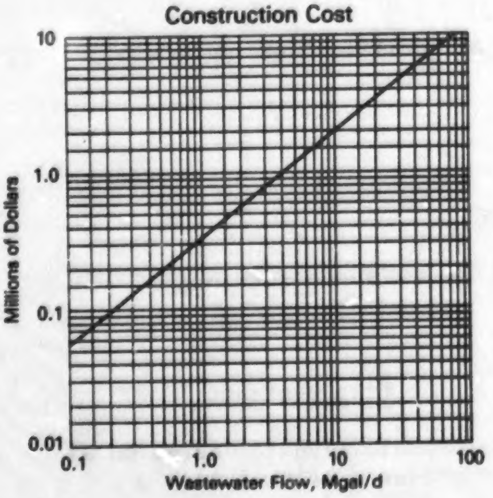
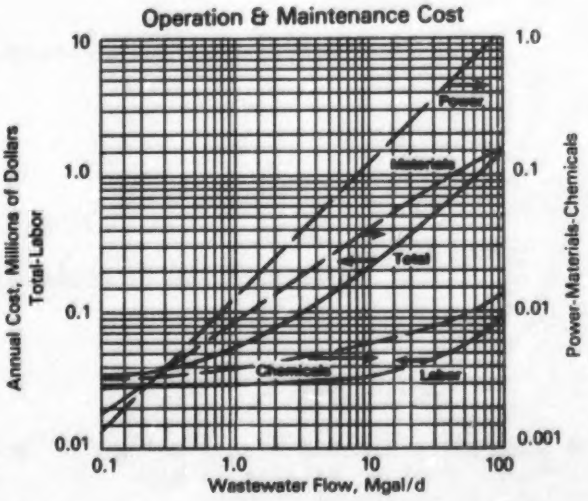


FIGURE B-41

OPERATION AND MAINTENANCE COSTS OF AMMONIA STRIPPING SYSTEM, EXCLUDING COST OF pH ADJUSTMENT; LABOR FIXED @ 59/HOUR, POWER @ 50.02/kwh.



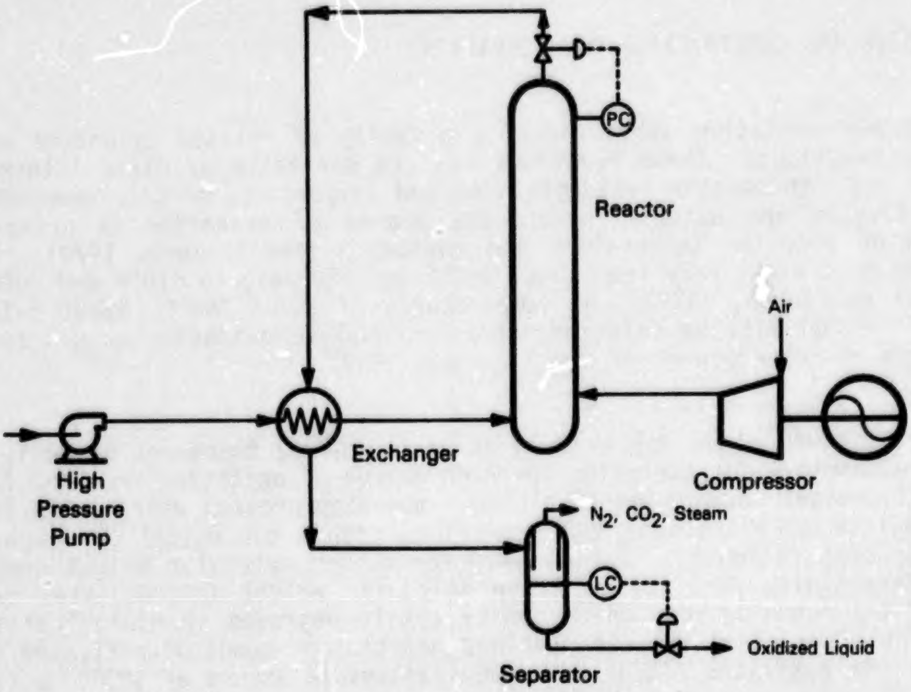
8.0 WET AIR OXIDATION

8.1 GENERAL DESCRIPTION AND APPLICATIONS

Wet air oxidation is based on the concept that any substance capable of burning can be oxidized in the presence of air at elevated pressure and moderate temperature. As Figure B-42 illustrates, wastewater containing some oxidizable material is mixed with air and pumped through an exchanger. It is then pumped to a reactor, where oxygen in air reacts with organic matter in wastes.

FIGURE B-42

WET AIR OXIDATION FLOW SCHEMATIC


PC
Reactor
Air
Exchanger
Compressor
High Pressure Pump
N_2, CO_2, Steam
LC
Oxidized Liquid
Separator

Oxidation is accompanied by temperature rise, and the heat produced can be used to sustain the process. After the reaction phase, gas and liquid are separated and liquid is used to heat the incoming material (Pradt, 1976).

Wet Air Oxidation (WAO) may be applied to a wide class of wastewaters such as those from the manufacture of pesticides, petrochemicals, pharmaceuticals, or other industrial chemicals (Zimpro, 1979). The process generally treats wastes with a high COD, ranging from 5-150 g/ℓ and, in general, is only suitable for high strength wastes.

The performance of WAO has been well demonstrated for the treatment of acrylonitrile wastes with high concentrations of cyanide and for the treatment of scrubbing liquor from coke oven gas clean-up, which contains cyanides, thiocyanates, and thiosulfates. Treatment of these wastes is currently practiced on an industrial scale (Zimpro 1979).

WAO has also been shown to effectively oxidize a number of toxic organics on a bench-scale level. Table B-31 shows the results of 1 hour WAO of 10 organics of temperatures of 320°C and 275°C. The maximum operating conditions for most installations is 275°C; most of the wastes were effectively treated at this temperature, although a catalyst may be required to achieve the desired degree of oxidation.

8.2 DESIGN AND CONSTRUCTION CONSIDERATIONS

Wet air oxidation takes place by a family of related oxidation and hydrolysis reactions. These reactions lead to partially oxidized intermediate products and, if reactor residence time and temperature permit, eventually to carbon dioxide and water. Hence, the degree of oxidation is primarily a function of reaction temperature and residence time (Zimpro, 1979). Actual operating conditions vary from about 350°F and 350 psig to 610°F and 3000 psig (Wilhelmi and Knopp, 1979). At temperatures of about 300°F, about 5-10 percent of the COD will be oxidized. Nearly complete oxidation occurs for most substances at temperatures of 610°F (Pradt, 1979).

Although WAO alone may be used as the complete treatment process, it is often uneconomical for achieving the high degree of oxidation required for effluent discharge. Zimpro has developed a two-step process whereby WAO is used to accomplish 40-95 percent COD reduction, with a biological or biophysical polishing step following. During WAO, the higher molecular weight compounds are preferentially oxidized to lower molecular weight intermediates such as methanol and formaldehyde, which can be easily degraded in biological treatment. The biophysical process involves addition of powdered activated carbon (PAC) to the activated sludge in concentrations in excess of 10,000 to 20,000 mg/ℓ. PAC acts as a flocculating agent promoting settling, and as a toxic sink for substances that might have survived WAO. It also allows for higher MLVSS concentrations than normally allowed with activated sludge (Wilhelmi and Ely, 1979). A combination of low-pressure oxidation and biological treatment was shown to be effective for a Louisville, KY sewage treatment plant's toxic sludges (Wilhelmi and Ely, 1979).

TABLE B-31

WET AIR OXIDATION OF TOXIC ORGANICS

1-Hour Wet Oxidations

Compound	Starting concentration[1]	% Starting material destroyed		
		320°C	275°C	275°C/Cu[11]
Acenaphthene	7.0 g/l	99.96%	99.99%	
Acrolein	8.41 g/l	>99.96%*	99.05%	
Acrylonitrile	8.06 g/l	99.91%	99.00%	99.50%
2-Chlorophenol	12.41 g/l	99.86%	94.96%	99.88%
2.4-Dimethylphenol	8.22 g/l	99.99%	99.99%	
2.4-Dinitrotoluene	10.0 g/l	99.88%	99.74%	
1.2-Diphenylhydrazine	5.0 g/l	99.98%	99.98%	
4-Nitrophenol	10.0 g/l	99.96%	99.60%	
Pentachlorophenol	5.0 g/l	99.88%	81.96%	97.30%
Phenol	10.0 g/l	99.97%	99.77%	

[1] The concentration remaining was less than the detection limit of 3 mg/ℓ.

Source: Zimpro, 1979

WAO can be thermally self-sustaining, requiring no additional fuel on feed COD's as low as 15,000 ppm. By comparison, incineration requires concentrations of 300,000 to 400,000 ppm (Zimpro, 1979) to be self-sustaining. The following equation can be used to determine the maximum waste COD (g/ℓ) which can be oxidized by the previously selected air to waste ratio (Liptak, 1974).

$$\text{COD} = 27.8 \times (\text{ratio of air to waste})$$

8.3 ADVANTAGES AND DISADVANTAGES

Advantages and disadvantages of the WAO process are summarized in Table B.32

TABLE B-32

ADVANTAGES AND DISADVANTAGES OF WET AIR OXIDATION

Advantages	Disadvantages
● Process can handle high waste concentrations of 5-150 g/ℓ	● Stainless steel equipment leads to higher capital costs than for incineration
● Reactor can be thermally self-sustaining at COD concentrations of 15,000 ppm, thus reducing operating costs	● Additional heat source will be needed when organic load is less than 15,000 ppm COD
● Creates no air pollution problems	● Requires well trained operators

8.4 COSTS

The costs of WAO are proportional to the volume of the waste stream, the required pressure, and the amount of air and auxiliary steam required. No cost curves were available for the wet air oxidation process. However, Tables B-33, B-34, and B-35 provide an idea of the associated costs.

TABLE B-33

FEED WASTE CHARACTERISTICS ASSUMED FOR COST OF WET AIR OXIDATION

Flow, gal/min	20
m/s	0.000126
	mg/L
pH	1.0
COD	70,000
Chloride	32,000
Total solids	118,000
Ash	23,000
TOC	30,000
Calcium	400
Magnesium	40
Sulfur	500

Source: Wilhelmi, 1979

TABLE B-34

PERFORMANCE/OPERATING CRITERIA FOR WET AIR OXIDATION

Temperature, °C	279
COD reduction, %	80
Effluent COD, mg/L	14,000
Effluent BOD, mg/L	8,000
Effluent pH	1

Source: Wilhelmi, 1979

TABLE B-35

COST OF WAO (1979 DOLLARS)

	WAO
Installed capital cost, $ millions	1.8
Operating costs, $:	
Fuel	-
Power	56,000
Steam	-
Maintenance	45,000
Labor	- (same)
Additional treatment	
Surcharge	31,500
Chemicals	- (same)
Total Operating Cost, $	$132,000

Source: Wilhelmi and Knopp, 1979

9.0 CHLORINATION

9.1 GENERAL DESCRIPTION AND APPLICATIONS

Chlorine is widely used in wastewater treatment for disinfection, odor control, and BOD reduction. It combines with water-forming hypochlorous acid which, in turn, can ionize to the hypochlorite ion.

$$Cl_2 + H_2O \longrightarrow HCL + HOCL \mathrel{\substack{\xrightarrow{\text{pH } 8} \\ \xleftarrow[\text{pH } 7]{}}} H + OCL$$

The disinfection potential is related to the strong oxidizing properties of hypochlorous acid (HOCL) and to a lesser extent, hypochlorite ion (OCL) (Liptak, 1974).

9.2 DESIGN AND CONSTRUCTION CONSIDERATIONS

The effectiveness of wastewater chlorination depends upon pH, temperature, time of contact, degree of mixing, and presence of interfering substances (Liptak, 1974).

Temperature and pH: The extent to which hypochlorous acid ionizes to form the weaker oxidizing hypochlorite ion depends on pH and temperature. Figure B-43 illustrates the extent of ionization as a function of these two variables.

Since hypochlorous acid is a more powerful oxidant, it is desirable to maintain a pH of less than 7.5.

Time of Contact: The rate of bacterial kill increases with time of exposure to chlorine. A detention time of 15 to 30 minutes is generally required in a baffled closed tank (Liptak, 1974).

Mixing: Complete and uniform mixing of chlorine with wastewater is important to disinfection. Any measurable short-circuiting can be ruinous to the process efficiency and, therefore, tank shape mixing and proper baffling are critical (Culp, 1978).

It has also been shown that rapid initial mixing may be important; the residuals formed initially are apparently more bactericidal than compounds formed later (Culp, 1978).

Interfering Compounds: Chlorine reacts readily with ammonia in water to form chloramines, which are much less effective oxidizing agents. Chlorine also oxidizes ferrous iron, sulfides, and nitrates. Presence of these species increases the chlorine demand (i.e., the amount of chlorine that will combine with various chemicals before it begins to appear as free chlorine residual) and increases the required dosage (Liptak, 1974).

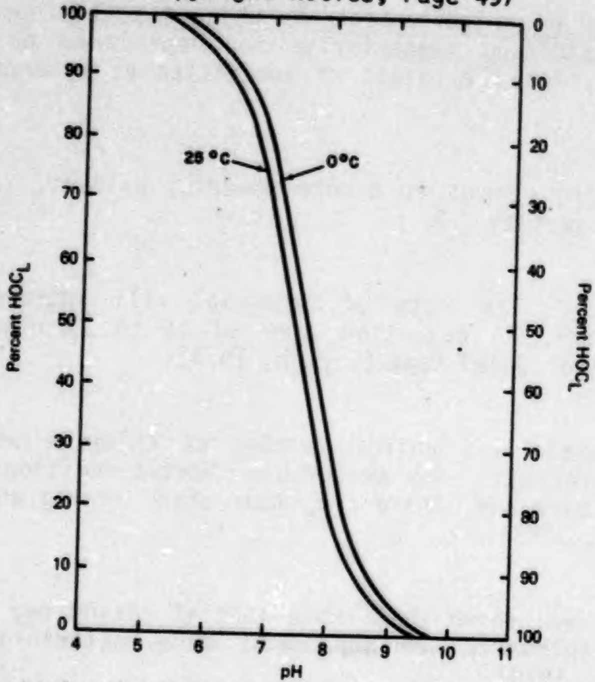
The size of the chlorinator is based on the chlorine demand of the water and on its flow rate. Typical chlorine dosages for disinfection are 3-15 mg/ℓ for trickling filter effluent and 2-8 mg/ℓ for activated sludge effluent (Metcalf and Eddy, 1972).

The chlorinator is equipped with a feed control system. The simplest and least expensive is a manual device for feed control. Automatic ratio control devices are available that can adjust the chlorine dosage to changes in flow rate. A more sophisticated control system can include a residual chlorine

FIGURE B-43

EXTENT OF HYPOCHLOROUS ACID (HOCl) IONIZATION INTO OCL^- AS A FUNCTION OF pH AND TEMPERATURE

(Source: Liptak, 1974)
See Copyright Notice, Page 497



analyzer, which controls the chlorine dosage based on residual chlorine levels (Liptak, 1974).

The most common chlorine compounds used in wastewater treatment are chlorine gas and calcium and sodium hypochlorite. The latter two are used in small treatment plants where simplicity and safety are more important than cost. Because calcium hypochlorite granules are readily soluble in water and are relatively stable under proper storage conditions, they are often favored (Metcalf and Eddy, 1972). Chlorine gas is risky to both store and use.

9.3 ADVANTAGES AND DISADVANTAGES

Table B-36 summarizes the advantages and disadvantages of chlorination.

TABLE B-36

ADVANTAGES AND DISADVANTAGES OF CHLORINATION

Advantages	Disadvantages
● Markedly reduces concentrations of harmful organisms	● May cause formation of chlorinated hydrocarbons
● Process is very reliable	● Chlorine gas is hazardous and requires careful handling
● Less expensive than alternative means of disinfection such as ozone	● Chlorine reacts with certain chemicals in water, leaving only the residual for disinfection

9.4 COSTS

Figures B-44 and B-45 show 1976 construction, operation, and maintenance costs for chlorination based on the parameters listed below:

COSTS -

1. Service life: 15 years
2. Equipment: Including chlorine supply, chlorinator, and contact chamber
3. Dosage = 10 mg/l; contact time = 30 minutes
4. Labor rate = $7.50/hr, including benefits
5. Power cost = $.02/kWh; chlorine cost = $160/ton
6. Index: ENR = 2475, September 1976.

FIGURE B-44

CONSTRUCTION COSTS FOR CHLORINATION

(Source: EPA, 1978)

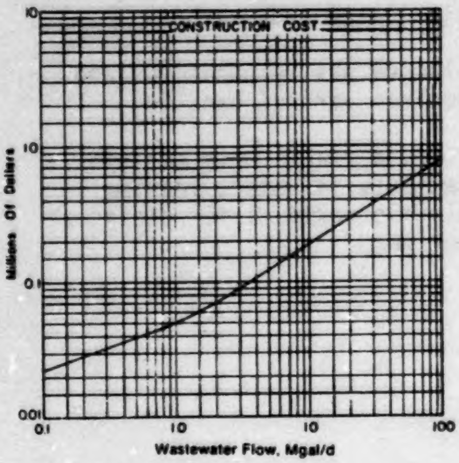
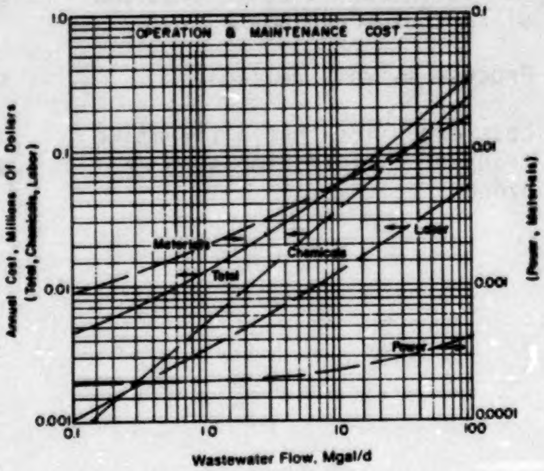


FIGURE B-45

O&M COSTS FOR CHLORINATION

(Source: EPA, 1978)



REFERENCES

Autotrol Corp. 1978. Autotrol wastewater treatment systems: design manual. Autotrol Biosystems Division. Milwaukee, WI.

Azad, Hardam (ed.). 1976. Industrial wastewater management handbook. New York: McGraw-Hill Book Co.

Culp, R.L. et al. 1978. Handbook of advanced wastewater treatment. New York: Van Nostrand Reinhold Company.

DeRenzo, D.H. 1978. Unit operations for treatment of hazardous industrial wastes. Noyes Data Corp., Park Ridge, NJ.

Dow Chemical Co. 1971. A laboratory manual on ion exchange. No. 177-1207-77.

Eckenfelder, W.W. 1963. Trickling filter design and performance. Transactions of the American Society of Civil Engineers 128 (Part III):371.

Ford, D.L. and L.F. Tischler. 1977. Guide to wastewater treatment. Chemical Engineering. August 1977.

Germain, J.E. 1966. Economical treatment of domestic waste by plastic-medium trickling filters. J. Water Pollution Control Federation 28:192.

Hammer, J. J. 1975. Water and wastewater technology. New York: John Wiley & Sons, Inc.

JRB Associates, Inc. 1980. Training manual for hazardous waste site investigations. Prepared for the U.S. Environmental Protection Agency. McLean, Virginia.

Krupka, M. May 1980. Personal communication with K. Wagner.

Liptak, B.G. (ed.). 1974. Environmental engineers' handbook, vol. 1: water pollution. Radnor, PA: Chilton Book Company.

Metcalf and Eddy, Inc. 1972. Wastewater engineering: collection, treatment, and disposal. New York: McGraw-Hill Book Company.

Nemerow, N.L. 1978. Industrial water pollution: origin, characteristics, and treatment. Reading, MA: Addison-Wesley Publishing Co.

Permutit. 1977. Sulfex heavy metals waste treatment process. Technical Bulletin 13:6.

Polybac Corporation. 1978. Phenobac mutant bacterial hydrocarbon degrader. New York, NY.

Pradt, L.A. 1972. Developments in wet air oxidation. Chemical Engineering Progress 68 (12):72-77. [Updated 1976.]

Rohm and Haas Company. 1979. Fluid process chemicals consumer price schedule. Independence Mall, Philadelphia.

Schulze, K.L. 1960. Conference on biological waste treatment. Manhattan College, April 1960.

U.S. Environmental Protection Agency. 1975. Process design manual for suspended solids removal. Technology Transfer Office, Washington, D.C. EPA-625/1-75-003a.

U.S. Environmental Protection Agency. 1977a. Demonstration of a leachate treatment plant. By Steiner, Keenan and Fungaroli for Office for Solid Wastes. Washington, D.C. PB-269-502.

U.S. Environmental Protection Agency. 1977b. Evaluation of leachate treatment, vol. 1: characterization of leachate; vol. 2: biological and physical/chemical processes. By E.S. Chian and F.B. Dewalle for Municipal Environmental Research Laboratory. EPA-600/2-77-186a.

U.S. Environmental Protection Agency. 1977. Wastewater treatment facilities for sewered small communities. Technology Transfer Division. Washington, D.C. EPA-625/1-77-009.

U.S. Environmental Protection Agency. 1978a. Innovative and alternative technology assessment manual. Office of Water Program Operations. EPA-430/9-78-009.

U.S. Environmental Protection Agency. 1978. State of the art report - pesticide disposal research. By Midwest Research Institute for Municipal Environmental Research Laboratory, Cincinnati, OH. EPA-600/2-78-183.

U.S. Environmental Protection Agency. 1979a. Estimating waste treatment costs, vol. 3: Cost curves applicable to 2,500 gpd to 1.0 mgd treatment plants. Municipal Environmental Research Laboratory, Cincinnati, OH. EPA-600/2-79-162c.

U.S. Environmental Protection Agency. 1979b. Selected biodegradation techniques for treatment and ultimate disposal of organic materials. Municipal Environmental Research Laboratory. EPA-600/2-79-006.

Velz, C.J. 1960. A basic law for performance of biological beds. Sewage Works Journal 20:245.

Vermuelen, T. 1977. Process arrangements for ion exchange and adsorption. Chemical Engineering Progress Vol. 73(10):57-61

Wilhelmi, A.R., and R.B. Ely. 1979. The treatment of toxic industrial wastewaters by a two-step process. 30th Annual Purdue Industrial Waste Conference.

Wilhelmi, A.R., and P.V. Knopp. 1979. Wet air oxidation--an alternative to incineration. Chemical Engineering Progress [Reprint].

Zimpro. 1979. Industrial pollution control systems. Zimpro Environmental Control Systems. Rothschild, WI.

Zitrides, T. 1978. Mutant bacteria for the disposal of hazardous organic wastewaters. Presented at: Pesticide disposal research and development symposium. Reston, VA., 6-7 September 1978.

APPENDIX C

COST INDICES (Average Per Year)

Year	Marshall & Stevens Installed Equipment Indices	Engineering News-Record Construction Index	Engineering News-Record Building Cost Index	Chemical Engineering Plant Construction Index	EPA Sewage Treatment Plant Construction Index
	1926=100	1913=100	1913=100	1957-1959=100	1957-1959=100
	All Industry				
1950	168	510	375	74	
1951	180	543	400	80	
1952	181	569	416	81	
1953	183	600	431	85	
1954	185	628	446	86	
1955	191	660	465	88	
1956	209	690	491	94	
1957	224	724	509	99	
1958	229	759	525	100	102
1959	235	797	548	101	104
1960	238	824	559	102	105
1961	237	847	568	101	106
1962	239	872	580	102	107
1963	239	901	594	102	109
1964	242	936	612	103	110
1965	245	971	627	104	112
1966	252	1,021	650	107	116
1967	263	1,070	672	110	119
1968	273	1,155	721	114	12[illegible]
1969	285	1,269	790	119	13[illegible]
1970	303	1,395	836	126	143
1971	321	1,581	948	132	160
1972	332	1,753	1,048	137	172
1973	344	1,895	1,138	144	182
1974	398	2,020	1,204	165	217
1975	444	2,212	1,306	182	250

--continued--

COST INDICES (Average Per Year) (continued)

Year	Marshall & Stevens Installed Equipment Indices	Engineering News-Record Construction Index	Engineering News-Record Building Cost Index	Chemical Engineering Plant Construction Index	EPA Sewage Treatment Plant Construction Index
	1926=100	1913=100	1913=100	1957-1959=100	1957-1959=100
	All Industry				
1976	472	2,401	1,425	192	262
1977	505	2,577	1,545	204	278
1978	545	2,776	1,674	219	305
1979	599	3,003	1,819	239	335
1980	660	3,159[1]	1,918[1]	261	358[2]
1981	721	3,705[1]	1,184[1]	273[1]	

[1] Based on December of year.
[2] Based on March of year.

COPYRIGHT NOTICE

Figure 3-4	From WATER RESOURCES ENGINEERING, 3rd ed. by Ray K. Linsley and Joseph B. Franzini, Copyright (c) 1979, by McGraw-Hill, Inc. Used by permission of McGraw-Hill Book Company.
Figure 4-11	From GROUNDWATER, by R. Allan Freeze and John A. Cherry, Copyright (c) 1979, p. 328. Reprinted by permission of Prentice-Hall, Inc., Englewood Cliffs, N.J.
Table 4-12	From GROUNDWATER, by R. Allan Freeze and John A. Cherry, Copyright (c) 1979, p. 318. Reprinted by permission of Prentice-Hall, Inc., Englewood Cliffs, N.J.
Figure 5-3 Figure 5-4	From "Love Canal aftermath: Learning from a tragedy," by R. S. Glaubinger, Excerpted by special permission from CHEMICAL ENGINEERING, Oct. 22, 1979, Copyright (c), by McGraw-Hill, Inc., New York, N.Y. 10020.
Figure 6-10	From "Pipe flow chart for turbulent flow." Reprinted by special permission from CHEMICAL & METALLURGICAL ENGINEERING, May 1937, Copyright (c) 1937, by McGraw-Hill, Inc., New York, N.Y. 10020.
Table 6-4	From CHEMICAL ENGINEERS' HANDBOOK, 5th ed. by Robert H. Perry and Cecil H. Chilton, ed. Copyright (c) 1973, McGraw-Hill, Inc. Used by permission of McGraw-Hill Book Company.
Table 6-6	From Table 6.29g, from THE ENVIRONMENTAL ENGINEERS HANDBOOK, Volume II, entitled Air Pollution and edited by Béla G. Lipták. Copyright 1974 by the authors. Reprinted with the permission of the publisher, CHILTON BOOK COMPANY, Radnor, PA.
Figure 7-2 Figure 7-4	From HANDBOOK OF HEAVY CONSTRUCTION, by F.W. Stubbs, Copyright (c) 1959, McGraw-Hill, Inc. Used by permission of McGraw-Hill Book Company.

COPYRIGHT NOTICE (Continued)

Figure 7-5	From GENERAL EXCAVATION METHODS, by A.B. Carson, Copyright (c) 1961, F.W. Dodge Corporation. Used by permission of McGraw-Hill Book Company.
Figure 7-7b	From WATER RESOURCES ENGINEERING by Ray K. Linsley and Joseph B. Franzini, Copyright (c) 1979, by McGraw-Hill, Inc. Used by permission of Ellicott Machine Corporation, Baltimore, Maryland.
Figure 7-9	From WATER RESOURCES ENGINEERING, by Ray K. Linsley and Joseph B. Franzini, Copyright (c) 1979, by McGraw-Hill, Inc. Used by permission of McGraw-Hill Book Company.
Table B-2 Table B-4 Table B-15	From INDUSTRIAL WASTEWATER MANAGEMENT HANDBOOK, by Hardam S. Azad, ed., Copyright (c) 1976 by McGraw-Hill, Inc. Used by permission of McGraw-Hill Book Company.
Table B-16	Reprinted from JOURNAL American Water Works Association, Volume 64, Number 6 (June, 1972), by permission. Copyright 1972, the American Water Works Association.
Figure B-34 Figure B-35	From CHEMICAL ENGINEERS' HANDBOOK by Robert H. Perry and Cecil H. Chilton, ed. Copyright (c) 1973, McGraw-Hill, Inc. Used by permission of McGraw-Hill Book Company.
Figure B-37 Figure B-38	From HANDBOOK OF ADVANCED WASTE WATER TREATMENT, 2nd ed., by Russel L. Culp et. al. Copyright (c) 1978 by Van Nostrand Reinhold Company. Reprinted by permission of the publisher.
Figure B-43	From Figure 5.23c, from THE ENVIRONMENTAL ENGINEERS HANDBOOK, Volume I, entitled Water Pollution and edited by Béla G. Lipták. Copyright 1974 by the authors. Reprinted with permission of the publisher, CHILTON BOOK COMPANY, Radnor, PA.

★ U.S. GOVERNMENT PRINTING OFFICE: [illegible] -559-092/3399